Multi-Dimensional Analysis of Large, Complex Slope Instability

by

Katherine S. Kalenchuk

A Thesis submitted to the Department of Geological Sciences and Geological Engineering In conformity with the requirements for the degree of Doctor of Philosophy (Ph.D.)

Queen’s University

Kingston, Ontario, Canada

(September 2010)

Copyright © Katherine S. Kalenchuk 2010
Abstract

Complex deformation processes observed in massive slow-moving, active, landslides are contributed to by topography, non-uniform shear surfaces, heterogeneous rockmass and shear zone strength characteristics, composite failure mechanisms and hydrogeology. This thesis provides a systematic means to account for geology, geomorphology and geomechanics when interpreting slope deformation processes. Significant contributions to the field of landslide geomechanics have been made by analyzing how spatially discriminated slope deformations are influenced by spatial variation of geological and geotechnical factors and temporal changes in piezometric levels.

The Downie and Beauregard landslides are massive instabilities that have extensive histories of slope monitoring and observational assessment, and where detailed site investigations have been completed. A methodology has been developed for the interpretation of 3-dimensional shear zone geometries using spatial prediction algorithms complemented by sound engineering judgment. The applicability of this process to other spatial data, such as displacement or piezometric records and measurements of material properties is demonstrated. Composite landslide deformations have been analyzed for both Downie and Beauregard to characterize global slope behaviour and identify localized events. Furthermore, a new interpretation of landslide morphological regions at Downie is provided.

The research presented in this thesis demonstrates the importance and value of 3-dimensional numerical modelling. A rigorous procedure to numerically simulate large landslides has been developed. This sophisticated method accounts for complex
geometries, heterogeneous shear zone strength parameters, internal shears, interaction between discrete landslide zones and piezometric fluctuations. This advance in state-of-the-art landslide modelling provides an important tool for investigating dynamic landslide systems.

Based on Downie and Beauregard field data numerical models have been calibrated to reproduce observed slope behaviour. The calibration process has provided insight on key factors controlling massive slope mechanics. Calibrated models are used to investigate how trigger scenarios may accelerate deformations at Downie and the effectiveness of a proposed slope drainage system at Beauregard. The ability to reproduce observed behaviour and forward test hypothesized changes to boundary conditions has valuable application in landslide hazard management. The capacity of decision makers to interpret large amounts of data, respond to rapid changes in a system and understand complex slope dynamics has been enhanced.
Co-Authorship

This thesis is based on the original work of Katherine Kalenchuk. Notable credit is given to the significant scientific and editorial contributions made by Jean Hutchinson and Mark Diederichs. The full references, including author list, for each submitted publication is included in Chapter 10: Conclusions.
Acknowledgements

This work has been made possible through financial contributions by NSERC (Natural Sciences and Research Council of Canada), CFI (Canadian Foundation for Innovation) and GEOIDE (GEOmatics for Informed DEcisions). A very important thank you goes to BC Hydro, particularly John Psutka and Dennis Moore, for Downie site and data access, and to Giovanni Barla, Marco Barla and Giovanna Piovano for Beauregard data access.

I would like to express my utmost appreciation for having incredible thesis advisors and mentors, Jean Hutchinson and Mark Diedrichs who have inspired and motivated me. This work would never have been possible without the brilliance, commitment and generosity of these phenomenal individuals.

A very special thank-you to Marlène Villeneuve, Maureen White, Matt Lato, Matt Perras and everyone else in our geomechanics research group; a priceless network of great friends who have never hesitated to set their own work aside when I needed to bounce around a few ideas.

My parents have taught me that there is more to life than work. I owe to them my pursuit of a career that I truly love.

This Ph.D. would not have been possible without the immeasurable love and support of my amazing husband, Colin Hume. And of course our little Sheldon has given me the most wonderful reason to sit back and enjoy the more important moments in life.
Statement of Originality

I hereby certify that all of the work described within this thesis is the original work of the author. No part of this thesis has been submitted elsewhere for any other degree or qualification. Any published (or unpublished) ideas and/or techniques from the work of others are fully acknowledged in accordance with the standard referencing practices.

Katherine S. Kalenchuk

(September 2010)
# Table of Contents

**ABSTRACT**

**CO-AUTHORSHIP**

**ACKNOWLEDGEMENTS**

**STATEMENT OF ORIGINALITY**

**TABLE OF CONTENTS**

**LIST OF FIGURES**

**LIST OF TABLES**

**LIST OF VARIABLES**

**LIST OF ACRONYMS**

**CHAPTER 1 GENERAL INTRODUCTION**

1.1 Goals of the Thesis

1.2 A Brief History of Massive Landslides

    1.2.1 Vaiont

    1.2.2 Frank Slide

    1.2.3 Goldau Rockslide

    1.2.4 La Clapière Landslide
1.2.5 Downie Slide

1.2.6 Beauregard Landslide

1.3 Landslide Types

1.4 Numerical Modelling of Landslides

1.5 Synopsis of Findings and Major Scientific Contributions

1.5.1 Interpretation of Spatial Data to Generate Three-Dimensional Shear Surface Geometries

1.5.2 Temporal Interpretations of Hydrogeology

1.5.3 Definition of Morphological Zones at Downie Slide and Interpretation of Slope Behaviour

1.5.4 Fusion of Monitoring Data Sources and the Analysis of Beauregard Slope Deformation Patterns

1.5.5 Development of Numerical Modelling Methodology

1.5.6 Advances to Landslide Geomechanics by Numerical Simulation of Observed Slide Behaviour

1.6 Thesis Format

1.7 References

CHAPTER 2 APPLICATION OF SPATIAL PREDICTION TECHNIQUES TO DEFINING THREE-DIMENSIONAL LANDSLIDE SHEAR SURFACE GEOMETRY
CHAPTER 3 THREE-DIMENSIONAL NUMERICAL SIMULATIONS OF THE DOWNIE SLIDE TO TEST THE INFLUENCE OF SHEAR SURFACE GEOMETRY AND HETEROGENEOUS SHEAR ZONE STIFFNESS
3.2 Introduction

3.3 Downie Slide Case Study

3.3.1 Shear Zone Geometry

3.3.2 Shear Zone Character

3.3.3 Slide Behaviour

3.4 Numerical Modelling

3.4.1 Model Geometry

3.4.2 Meshing

3.4.3 Material Properties and Constitutive Models

3.4.4 Boundary Conditions

3.4.5 Pore Pressure

3.4.6 Buttress Load of a Full Reservoir

3.4.7 In-Situ Stresses

3.4.8 Damping

3.4.9 Numerical Procedure

3.4.10 Comparison of Field and Simulated Data

3.5 Results and Discussion

3.5.1 Shear Geometry

3.5.2 Spatial Variation in Joint Normal and Shear Stiffness
CHAPTER 4 MORPHOLOGICAL AND GEOMECHANICAL ANALYSIS
OF THE DOWNIE SLIDE USING 3-DIMENSIONAL NUMERICAL
MODELS: TESTING THE INFLUENCE OF INTERNAL SHEAR ZONES
AND INTERACTION BETWEEN LANDSLIDE REGIONS ON
SIMULATED SLOPE BEHAVIOUR

<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.1</td>
<td>Abstract</td>
<td>131</td>
</tr>
<tr>
<td>4.2</td>
<td>Introduction</td>
<td>132</td>
</tr>
<tr>
<td>4.3</td>
<td>Case Study</td>
<td>133</td>
</tr>
<tr>
<td>4.4</td>
<td>Landslide Zoning</td>
<td>139</td>
</tr>
<tr>
<td>4.4.1</td>
<td>Landslide Boundary</td>
<td>140</td>
</tr>
<tr>
<td>4.4.2</td>
<td>The Upper Region</td>
<td>142</td>
</tr>
<tr>
<td>4.4.3</td>
<td>South Trough</td>
<td>145</td>
</tr>
<tr>
<td>4.4.4</td>
<td>Talus Slopes</td>
<td>147</td>
</tr>
<tr>
<td>4.4.5</td>
<td>Central Region</td>
<td>147</td>
</tr>
<tr>
<td>4.4.6</td>
<td>Lower Region</td>
<td>150</td>
</tr>
<tr>
<td>4.4.7</td>
<td>North Knob</td>
<td>158</td>
</tr>
<tr>
<td>4.4.8</td>
<td>Over-Steepened Slopes</td>
<td>160</td>
</tr>
</tbody>
</table>
4.4.9 Toe Slump

4.4.10 Basin

4.4.11 North Disturbed Zone

4.4.12 Lobe

4.4.13 Slide Behaviour

4.5 Numerical Modelling

4.5.1 Model Development

4.5.2 Comparison of Field vs. Simulated Data

4.6 Results and Discussion

4.6.1 Intermediate Shear Zones

4.6.2 Boundary Adjustments

4.7 Summary

4.8 Acknowledgements

4.9 References

CHAPTER 5 DOWNIE SLIDE - SENSITIVITY OF WATER TABLE INTERPRETATIONS TO TEMPORAL CHANGES IN DATA DISTRIBUTION

5.1 Abstract

5.2 Introduction

5.3 Water Table Interpolation
5.3.1 Artifacts of Changing Data Sets 202

5.3.2 Extrapolation of Piezometric Records 204

5.4 Results and Discussion 209

5.5 Summary 211

5.6 Acknowledgments 213

5.7 References 213

CHAPTER 6 DOWNIE SLIDE – NUMERICAL SIMULATION OF GROUNDWATER FLUCTUATIONS INFLUENCING BEHAVIOUR OF A MASSIVE LANDSLIDE 216

6.1 Abstract 216

6.2 Introduction 217

6.3 Case Study 219

6.4 Groundwater Conditions 222

6.4.1 Interpreting Water Tables from Piezometric Data 222

6.4.2 Temporal Variation in Groundwater Conditions 224

6.4.3 Seasonal Variation in Groundwater Conditions 230

6.5 Slide Behaviour 232

6.6 Numerical Modelling 236

6.6.1 Numerical Procedure 240

6.6.2 Comparison of Field and Simulated Data 244
CHAPTER 7 INTEGRATION OF THREE SOURCES OF SLOPE MONITORING DATA TO CHARACTERIZE LANDSLIDE BEHAVIOUR, BEAUREGARD LANDSLIDE, NORTHWESTERN ITALY

7.1 Abstract 256
7.2 Introduction 257
7.3 Case Study 258
7.4 Data Description 260
7.5 Data Integration 265
7.6 Conclusions 271
7.7 Acknowledgements 272
7.8 References 272

CHAPTER 8 A GEOMECHANICAL ANALYSIS OF THE BEAUREGARD LANDSLIDE USING THREE-DIMENSIONAL NUMERICAL MODELS

8.1 Abstract 274
8.2 Introduction 275
8.3 Background 277
8.4 Geological, Structural and Morphological Setting 279
8.5 Groundwater 282
8.6 Landslide Monitoring and Slope Behaviour 285
  8.6.1 Temporal Variations in Slope Behaviour 287
8.7 Numerical Modelling 288
  8.7.1 Landslide Geometry 289
  8.7.2 Constitutive Models and Material Properties 293
  8.7.3 Interaction between Landslide Zones 298
  8.7.4 Drainage Testing 299
8.8 Summary 303
8.9 Acknowledgments 305
8.10 References 305

CHAPTER 9 SUMMARY AND DISCUSSION 309

9.1 Site Characterization 309
  9.1.1 Three-Dimensional Interpretation of Shear Zone Geometry 309
  9.1.2 Temporal Interpretations of Groundwater Levels 312
  9.1.3 Heterogeneous Shear Zone Parameters 313
9.2 Interpreting Slope Deformation and Identifying Morphological Zones 314
9.2.1 Improved Hazard Management by Understanding Composite Behaviour

9.2.2 Optimizing Slope Monitoring Practices

9.2.3 Deformation Patterns Common to Massive Slow Moving Landslides and Mechanisms of Rapid Failure

9.3 Development of Numerical Modelling Methodology

9.4 Numerical Simulation of Observed Slide Behaviour

9.4.1 Application of Numerical Simulations to Landslide Hazard Management - Testing Mitigation Techniques

9.4.2 Application of Numerical Simulations to Landslide Hazard Management - Testing Trigger Scenarios

9.5 References

CHAPTER 10 CONCLUSIONS

10.1 Downie Slide

10.1.1 Landslide Site Characterization

10.1.2 Slope Behaviour

10.1.3 Numerical Modelling

10.2 Beauregard Landslide

10.2.1 Landslide Site Characterization

10.2.2 Slope Behaviour
10.2.3 Numerical Modelling 333

10.3 Geomechanical Numerical Modelling of Massive Landslides 334

10.4 Limitations of Presented Research 336

10.5 Recommendations and Future Areas of Research 337

10.6 Contributions 338

10.6.1 Articles Published in Refereed Journals 339

10.6.2 Articles Submitted to Refereed Journals 339

10.6.3 Invited Book Chapters 340

10.6.4 Invited Papers 340

10.6.5 Fully Refereed Conference Papers (Paper reviewed and published in proceedings) 340

10.6.6 Partially Refereed Conference Papers (Abstract Reviewed – Paper Published in Proceedings) 341

10.6.7 Published Abstracts 341

10.6.8 Posters and Presentations 342

10.7 References 343

APPENDIX A PREDICTED GEOMETRIES FOR CONTINUOUS SHEAR SURFACES 345

APPENDIX B 3DEC MODELLING 350

B.1 Model Theory 350
B.1.1 Block Interaction 350
B.1.2 Sub-contact Force Update 351
B.1.3 Continuum Behaviour 352

B.2 Model Code 353
B.2.1 Master File 354
B.2.2 Build Geometry 354
B.2.3 Cut Shear Interfaces 356
B.2.4 Generate Continuum Mesh 363
B.2.5 Set-up Instruments 364
B.2.6 Set-up Rockmass Pore Pressures 386
B.2.7 Parameter Initialization and Set-up Sequence 387
B.2.8 Time Step Model 398
B.2.9 Load Contact Pore Pressure 399
B.2.10 Re-Load Contact Pore Pressure 401
B.2.11 Vary Joint Stiffness 403
B.2.12 Check for Steady State 404
B.2.13 Check for exploding nodes 405
B.2.14 Data Files 407

B.3 Model Sensitivity Testing 410

xvii
B.3.1 Grid Resolution and Mesh Density Sensitivity Testing 410

B.3.2 Sensitivity Testing of Downie Slide Shear Surface Strength Parameters (Friction and Cohesion) 411

B.4 References 412
### List of Figures

**Figure 1.1:** The Vaiont Landslide as viewed looking south from across the valley, inset shows location in northeastern Italy (photograph taken in May, 2008). 

**Figure 1.2:** Map of the Vaiont Valley (modified from Kiersch and Asce 1964).

**Figure 1.3:** Before and after photographs of Longarone looking up the valley towards the dam. (Photographs have been taken in May, 2008 of images on display at Visitor Center in Longarone).

**Figure 1.4:** Geological interpretations of Section AA’ (See Figure 1.2 for section location) (top: Kiersch and Asce 1964, middle: Kiersch 1988, bottom: Semenza and Ghirotti 2000 after Ghirotti 1993).

**Figure 1.5:** History of precipitation, reservoir level, displacement rates and piezometric levels between 1960 and 1963 (based on Müller 1987).

**Figure 1.6:** Frank Slide as viewed today looking west, inset shows location in southwestern Alberta, Canada (photograph taken in August, 2008).

**Figure 1.7:** Typical cross-section through Turtle Mountain (1) Banff Formation, (2) Livingstone Formation, (3) Fernie Group, (4) Kootenay Formation, (5) Blairmore Group (Cruden and Krahn 1978).

**Figure 1.8:** (top) LiDAR (Light Detection And Ranging) data of the Turtle Mountain area, (bottom) close up of the mountain crest illustrating visible tension cracks (data courtesy of Froese and Moreno 2005).

**Figure 1.9:** (top) The Goldau Rockslide source area, (lower right) side scarp (lower left) head scarp, and (inset) location of Goldau in central Switzerland (photographs taken in May, 2008).

**Figure 1.10:** The strata at Goldau are made up of weak marl units overlain by more competent massive conglomerate. (bottom) Photographs showing the nature of the conglomerate and slicken lines (left) on the conglomerate contact which indicate progressive creep (photograph taken in May, 2008). Figure 1.9 illustrates the photograph source location where bedding units have been exposed by localized recent (2002) instability.

**Figure 1.11:** Photograph of La Clapière as viewed from the south side of the valley, inset shows location in southeastern France (photograph taken in May, 2008).
Figure 1.12: Map and cross-section illustrating geology, structure and morphology of the La Clapière landslide (based on Cappa et al. 2004, Gunzburger and Laumonier 2002).

Figure 1.13: Overview photo of Downie Slide as viewed from the east side of the valley, inset shows location in southeastern British Columbia, Canada (Photograph taken in August, 2008).

Figure 1.14: Regional stratigraphy and structure of Downie Slide (from Brown and Psutka 1980).

Figure 1.15: Morphological regions of Downie Slide based on Patton and Hodge (1975) and Piteau et al. (1978).

Figure 1.16: Beauregard Landslide as viewed from the east side of the valley, with inset map showing location in northwestern Italy (photograph taken in September, 2009).

Figure 1.17: Morphological regions identified by Barla et al. (2006), Zone 1: Glacial morphology, Zone 2: Fractured rockmass features ridges and trenches and accumulation of debris, bounded by the mountain ridge and main head scarp and is interpreted as a stable portion of the landslide. Zone 3: Active portion of the instability, inset shows DEM of landslide slope.

Figure 1.18: Zones of maximum shear strain rate resolved using FLAC (Barla et al. 2006) (left) slope toe, (right) overall slope.

Figure 2.1: Location map of Downie Slide located in southeastern British Columbia, Canada.

Figure 2.2: Aerial photograph of Downie Slide illustrating the landslide boundary and the location of sub-surface data points.

Figure 2.3: (top) View of Downie Slide looking west, (lower left) the headscarp, (lower right) folding of the interlayered schists, gneisses and quartzites observed in drainage adit tunnel located in central lower portion of the slide.

Figure 2.4: Geological logging records and core photographs for drillhole S8 provided by BC Hydro demonstrate where boreholes intersect shear surfaces, and also indicate shear zone thickness. The shear zone is evident in core photos by increased fracture frequency and zones of rubble and gouge between the 645 and 708 ft. markers. Core runs are 1.5 m long.

Figure 2.5: Inclinometer data for two boreholes demonstrating a shear zone (B2) that is noted in the geological data logs, with a very obvious
displacement horizon in the inclinometer profile, shear zones (A2 and B1) that are noted in the geological data logs, but are not apparent in the inclinometer data, and potential shears (A1 and A3) that are apparent in the inclinometer data but not noted in the geological data logs.

Figure 2.6: Assumptions of a smooth, continuous shear surface (top) and a stepped, discontinuous shear surface (bottom).

Figure 2.7: Geometries (looking northwest) and contour plots of the interpolated lower shear at Downie Slide illustrating the error values at sub-surface data points returned from cross-validation for (1) continuous surfaces (a) minimum curvature (internal tension = 0, boundary tension = 1), (b) kriging of a variogram model, (c) the multiquadratic radial basis function, (d) the thin-plate-spline function, (2) discontinuous surface (e) minimum curvature, and (3) simplified surface (f) the elliptical parabola.

Figure 3.1: (top left) Location map of Downie Slide located in southeastern British Columbia, Canada (modified after Kalenchuk et al. 2009a – Chapter 2), (top right) schematic of Downie Slide geological setting and morphological regions (Kalenchuk et al. 2010a - Chapter 4 after Piteau et al. 1978, Brown and Psutka 1980, Patton and Hodge 1975), (bottom) morphological zoning defined by distinct morphological features and specific slope behaviour (Kalenchuk et al. 2010a – Chapter 4).

Figure 3.2: (top) Three-dimensional geometries (looking northwest) of the Downie Slide basal slip surface for (1) continuous (a. continuous minimum curvature (smooth), b. kriging of a variogram model and c. the multiquadratic radial basis function), (2) stepped (d. minimum curvature (discontinuous)), and (3) simplified (e. the elliptical parabola) interpretations (modified from Kalenchuk et al. 2009a – Chapter 2). (bottom) Exploded view of a three-dimensional Downie Slide model.

Figure 3.3: Contoured deformation rate standard deviations about the global mean; higher rates of movement occur near the head scarp and the central toe of the slide, while the middle portion of the slope is characterized by lower velocities (modified after Kalenchuk et al. 2010a – Chapter 4).

Figure 3.4: Displacement vectors scaled according to annual displacement rates averaged over the operating life of the Revelstoke Reservoir (data available 1985-2003).

Figure 3.5: Development of landslide model topography. From left to right: one column, height defined by topographic elevation and column width
defined by user specified grid resolution, several columns and a completed topographic model.

Figure 3.6: To define the local elevation and orientation of shear surfaces, each column is subdivided into sub-columns which are independently cut by joints.

Figure 3.7: Downie Slide model (a) looking northwest, (b) typical W-E cross section (Kalenchuk et al. 2009b).

Figure 3.8: Mohr-Coulomb failure criterion in 3DEC (after Itasca Consulting Group, Inc. 2003).

Figure 3.9: Conceptual schematic of the hybrid continuum-discontinuum landslide modelling (modified from Kalenchuk et al. 2009b).

Figure 3.10: Multiple water tables have been identified at Downie Slide (Kalenchuk et al. 2009c). For numerical simulation, the lower water table (confined below the basal slip surface) is applied to the basal slip surface and material below the basal slip surface, the upper water table is applied to material within the landslide.

Figure 3.11: A toe load is applied to account for the buttressing effect of the reservoir.

Figure 3.12: (top) Displacement rate standard deviation from the mean measured in the field and modelled returns R2 values used to quantitatively compare various model simulations (bottom) Contoured displacement rate standard deviations for visual comparison between modelled and measured data.

Figure 3.13: Contoured deformation rates measured in numerical models at the location of field survey monuments. (1) Continuous (a. minimum curvature, b. kriging of a variogram model and c. the multiquadratic radial basis function), (2) stepped (d. minimum curvature), and (3) simplified (e. the elliptical parabola) geometries.

Figure 3.14: Statistical correlation using R2 values to compare displacement rate standard deviation from the mean measured in the field versus modelled geometries.

Figure 3.15: Schematic illustrating that the stepped surface geometry has a shallower grade than the continuous shear surface geometries through the upper portion of the slope (inset shows section location).

Figure 3.16: Cosine values depicting correlation in direction of deformation between field and simulated data points for (1) continuous (a. minimum curvature (smooth), b. kriging of a variogram model and c.
the multiquadratic radial basis function), (2) stepped (d. minimum curvature (discontinuous)), and (3) simplified (e. the elliptical parabola) geometries.

Figure 3.17: (left) Distribution of shear zone thickness defined from borehole logs. (right) Thickness regions are used to define variation in contact stiffness parameters.

Figure 3.18: Contoured deformation rates for the continuous minimum curvature shear surface geometry at varying ratios of stiffness parameters (a) 0.1:1:10, (b) 0.2:1:5, (c) 0.5:1:2, (d) 1:1:1, (e) 2:1:0.5, (f) 5:1:0.2, (g) 10:1:0.1.

Figure 3.19: Contoured deformation rates for the continuous krigging shear surface geometry at varying ratios of stiffness parameters (a) 0.1:1:10, (b) 0.2:1:5, (c) 0.5:1:2, (d) 1:1:1, (e) 2:1:0.5, (f) 5:1:0.2, (g) 10:1:0.1.

Figure 3.20: Contoured deformation rates for the stepped minimum curvature shear surface geometry at varying ratios of stiffness parameters (a) 0.1:1:10, (b) 0.2:1:5, (c) 0.5:1:2, (d) 1:1:1, (e) 2:1:0.5, (f) 5:1:0.2, (g) 10:1:0.1.

Figure 4.1: Schematic of Downie Slide geological setting (based on Piteau et al. 1978, Brown and Psutka 1980) and morphological regions (based on Patton and Hodge 1975, Piteau et al. 1978). Inset map of Canada shows study area situated in southeastern British Columbia.

Figure 4.2: Morphological zones at Downie Slide defined according to distinct morphological features and specific slope behaviour.

Figure 4.3: The geometry of Downie head and side scarps; (top) view of the scarps looking west, (lower) blocky nature of the scarp face, (inset) stereonet illustration of joint sets (modified after Kalenchuk et al. 2009a).

Figure 4.4: LiDAR image of Downie Slide (left) plan view illustrating side and head scarps and the north landslide boundary (dash line) clearly visible in LiDAR data (right) isotropic views looking northwest (top) and southwest (bottom).

Figure 4.5: LiDAR image of the upper region clearly shows hummocky terrain.

Figure 4.6: Inclinometer S08 shows deformation distributed across the lower shear zone and also through the landslide mass. Dark and light grey horizons (in all inclinometer figures) mark the basal and internal shear zones respectively, as recorded in borehole geology logs.

Figure 4.7: Inclinometer S51 shows deformation distributed across the lower shear zone, through the landslide mass and also within the secondary shear zone.
Figure 4.8: (top) Scarp feature within the south trough, (bottom) overgrown jumbled talus. Inset aerial photographs show locations where photographs were taken (modified after Kalenchuk et al. 2009a) 146

Figure 4.9: LiDAR image of the central region shows varying morphology between the north- and south-central regions. 148

Figure 4.10: Inclinometer S23 shows surficial activity with minor displacements on the basal shear and insignificant movement through the landslide mass. 149

Figure 4.11: Inclinometer S03 shows slip on upper internal shear zone, with negligible deformation through landslide mass or along the lower internal shear surface. This inclinometer does not extend deep enough to intersect the basal shear zone. 149

Figure 4.12: LiDAR image of lower region. The broad ridge of the south-lower region shows east-west trending extensional lineaments, the north-lower region features the north and south lobes of the active zone, and toe sloughage is evident along the reservoir. 151

Figure 4.13: Inclinometer S02 shows shearing on the basal slip surface and some back-rotated deformation through the landslide mass. 152

Figure 4.14: Inclinometer S12 shows slightly back-rotated deformation through the landslide mass, with no displacements isolated along shear zones. 152

Figure 4.15: Inclinometer S14 shows shearing on the basal slip surface and some back-rotated deformation through the landslide mass. Noisy data above the internal shears may indicate a more disturbed portion of the slide profile. 153

Figure 4.16: Inclinometer S30 shows shearing on the basal slip surface as well as considerable surficial deformation. 153

Figure 4.17: Inclinometer S01 does not extend deep enough to provide a profile through the entire landslide mass, deformation though the upper portion of the mass is evident. 154

Figure 4.18: Inclinometer S07 shows no slip on the lower shear and deformation through the landslide mass (according to 25/06/79 data only). Between 1979 and 1984 the borehole was blocked and more recent data provides deformation data above 130 m depth only. 155

Figure 4.19: Inclinometer S17 shows deformation through the landslide mass with no displacements specifically isolated along internal shear zones. This inclinometer does not extend deep enough to intersect the basal shear zone. Surficial deformation associated with toe sloughing is evident. 155
Figure 4.20: Inclinometer S43 shows minor slip on both the basal and internal shear surfaces, more so along the latter, with some deformation through the landslide mass, particularly below the internal shear.  

Figure 4.21: Inclinometer S44 shows deformation through the landslide mass.  

Figure 4.22: Inclinometer S47 does not extend deep enough to span the entire landslide profile. The available data shows surficial deformation associated with the toe slough region.  

Figure 4.23: Photographs taken in the depletion zone at the boundary between the north-middle and north-lower regions. (top left) Valley parallel trough, (bottom left) internal scarp, (right) sink hole. Inset shows area where photographs were taken.  

Figure 4.24: (upper) North knob viewed from the north, (middle) tension crack located southeast of the knob, and (lower) extensional feature observed near the boundary of the north knob and the basin (red arrows indicate direction of extension) (modified after Kalenchuk et al. 2009a). Inset shows location where photographs were taken.  

Figure 4.25: Inclinometer S09 shows deformation through the landslide mass, directed southeast towards the active area and no displacements are specifically isolated along internal shear zones. This inclinometer does not extend deep enough to intersect the basal shear zone.  

Figure 4.26: LiDAR image of the north landslide toe showing the over-steepened slopes and toe slump regions located north and east of the north knob.  

Figure 4.27: Photographs taken along the boundary of the active north toe area; (left) scarp exposure, (right) magnitude of offset at the side boundary of the active area (modified after Kalenchuk et al. 2009a). Inset shows photograph source area.  

Figure 4.28: Inclinometer S32 shows significant surficial deformation in the toe slump region with minor displacement on the lower shear and negligible deformation through the landslide mass.  

Figure 4.29: LiDAR image of the basin region highlighting the trough and ridge morphology as well as extension along the north basin boundary.  

Figure 4.30: Photographs depicting extensional features at the north basin boundary, inset shows location where photographs were taken.  

Figure 4.31: Inclinometer S50 shows very minor deformation though the landslide mass in the basin region with no evidence of slip along shear surfaces.  

Figure 4.32: LiDAR image of the lobe and disturbed north zones.
Figure 4.33: Contour plots of displacement rate standard deviation from the mean measured between 1990 and 2003; (a) all survey data utilized, (b) anomalous toe slump data is removed and (c) all data points with significant surficial deformation is removed to best demonstrate the true overall landslide behaviour.

Figure 4.34: Schematic illustrating material properties utilized in 3DEC simulations (modified after Kalenchuk et al. 2009b).

Figure 4.35: (left) Spatial variation in the thickness of the basal shear zone. (right) Thick, average and thin regions used to assign spatially varied stiffness parameters to shear surface in 3DEC simulations (Kalenchuk et al. 2010 – Chapter 3).

Figure 4.36: Comparison of various adjusted landslide boundaries to include or exclude specific landslide zones to create differing monolithic representations; (a) smoothed south (b) smoothed north (c) no knob (d) hypothesis. (left) Aerial photograph with dark long-hatch line showing boundary of monolithic mass and white short-hatch line showing adjusted geometry, (right) cross sections where solid grey line shows basal shear surface for the adjusted geometry and hatched line shows monolithic mass.

Figure 4.37: Two models with stepped minimum curvature basal slip surface geometry and varying interpretations of internal secondary shears are used as examples to illustrate the comparison of field and modelled data. (top) Displacement rate standard deviation from the mean measured in the field vs. modelled returns R2 values used to quantitatively compare various model simulations (bottom) Contoured displacement rate standard deviations for visual comparison.

Figure 4.38: Contoured deformation rate standard deviation about the mean measured in numerical models at the location of field survey monuments for varying basal shear surface geometries defined by (left) continuous minimum curvature algorithm, (center) continuous kriging function (right) discontinuous minimum curvature algorithm.

Figure 4.39: Cosine values depicting the correlation in direction of deformation between field and simulated data points for varying basal shear surface geometries.

Figure 4.40: Contoured deformation rates measured in numerical models at the location of field survey monuments for various adjustments to landslide boundary geometries.

Figure 4.41: Cosine values depicting the correlation in direction of deformation between field and simulated data points for various adjustments to landslide boundary geometries.
Figure 5.1: Downie Slide map illustrating local geological setting, topography and location of borehole piezometers and drainage adits (adit portals are at approx. 590 m a.s.l.), (inset) location map of the Downie Slide in southeastern British Columbia, Canada.

Figure 5.2: Cross-section AA’ illustrating multiple water tables identified at Downie Slide; they are confined by the low permeability of landslide shear surfaces.

Figure 5.3: Piezometer installation and decommission record, shaded regions show operating life of instrument in each borehole.

Figure 5.4: In 1993 a number of instruments were installed in boreholes S49, S48, S54, and S52 creating apparent changes in the ground water levels. These apparent changes in the level of the middle water table between 1992 and 1993 are artifacts of changes to data distribution.

Figure 5.5: Data from instrumented boreholes S31, S36, S37 and S38 demonstrate that there has been no change in ground water boundary conditions in the area proximal to S45 and S52 since the drainage system was built. Therefore it is reasonable to extrapolate S45 and S52 data backwards in time.

Figure 5.6: (top-left) Apparent change in ground water levels from 2001 to 2002 as S13 is taken offline, (top-right) change from 2001 to 2002 when S13 data is extrapolated forward. (bottom) Piezometric trends for boreholes near S13 show no change in boundary conditions after 2001; therefore it is reasonable to assume that S13 data can be extrapolated forward in time.

Figure 5.7: Extrapolation of S25/49 borehole piezometers to attain a complete middle water table record at the spatial location of these instruments.

Figure 5.8: Installation of borehole S49 influence of data interpretation between 1992 and 1993; (left) interpreted from raw data (right) interpreted from piezometric data where S49 data has been extrapolated through time.

Figure 5.9: Ground water fluctuations with changing boundary conditions at each stage of reservoir operations, including; (top) drainage development, (middle) reservoir filling, and (bottom) reduced drainage capacity; interpreted from raw data (left), and from piezometric data which has been extrapolated through time (right).

Figure 6.1: (top left) Location map of the Downie Slide in the Columbia River Valley in southeastern British Columbia, (top right) local geological setting and (bottom) aerial photograph of the Downie Slide showing
landslide zones based on landslide morphology and spatially discriminated slope behaviour (modified after Kalenchuk et al. 2010a – Chapter 3).

Figure 6.2: Schematic of a typical cross-section illustrating multiple water tables that have been identified at Downie Slide (Kalenchuk et al. 2009a). For numerical simulation, the lower water table (confined below the basal slip surface) is applied to the basal slip surface and material below the basal slip surface; the upper water table is applied to material within the landslide and internal secondary shears.

Figure 6.3: Cross-sections showing variations in water tables interpreted for different ground water states at Downie Slide, including pre-drainage, post-drainage, post-reservoir filling and late-reservoir life.

Figure 6.4: Water table changes (vertical m) between 1973 and 1983, due to drainage system development; interpreted using temporal data extrapolation (Kalenchuk et al. 2010c – Chapter 5); (top) middle water table, (bottom) lower water table.

Figure 6.5: Water table changes (vertical m) between 1983 and 1985 due to reservoir filling; interpreted using temporal data extrapolation (Kalenchuk et al. 2010c – Chapter 5); (top) middle water table, (bottom) lower water table.

Figure 6.6: Water table changes (vertical m) between 1985 and 2003 due to gradual losses in drainage system capacity over the Revelstoke Reservoir operating life; interpreted using temporal data extrapolation (Kalenchuk et al. 2010c – Chapter 5); (top) middle water table, (bottom) lower water table.

Figure 6.7: Spatial variation in the minimum magnitude of seasonal ground water fluctuations at Downie Slide (upper water table). Data from point A shows high seasonal fluctuations near the head scarp recharge zone, and data from point B shows very little annual variation (Kalenchuk et al. 2009a). The lower water table shows similar trends.

Figure 6.8: Deformation rates vary spatially and temporally, as different zones of the landslide show variable response to changing ground water boundary conditions (left) all instruments, and (right) surficial data is removed.

Figure 6.9: (top) Schematic illustrating modelling parameters, (center) typical cross-section through model showing in situ material, main landslide body and active zone, (lower) isotropic view of three-dimensional model looking northwest.
Figure 6.10: Stiffness parameters are applied to simulate shear surfaces in numerical models as a function of shear zone thickness. (top) Contour plot illustrating variation in thickness of the Downie Slide basal shear zone, (bottom) thick, average and thin regions identified for application of specific stiffness values (modified after Kalenchuk et al. 2010a – Chapter 3).

Figure 6.11: Cross-sections showing water tables interpreted to represent (top) rapid reservoir drawdown where late-reservoir life is applied with no buttressing load of the reservoir at the landslide toe and (bottom) total loss in drainage capacity under full reservoir operating conditions.

Figure 6.12: Modelled and measured normalized change in deformation rates through sequential changes to ground water levels over Revelstoke Reservoir development and operation.

Figure 6.13: Normalized change in deformation rates from numerically simulated response to hypothesized water table changes; rapid reservoir drawdown and total losses in drainage capacity.

Figure 7.1: Location map of Beauregard Landslide located in northwestern Italy, illustrating the distribution of Leica and GPS survey points.

Figure 7.2: Leica survey data collected at target location K17 with trend-lines to resolve displacement rates over the 2004-2009 time period.

Figure 7.3: Poor correlation between the magnitudes of seasonal fluctuations versus the nominal distance of survey targets from the Leica total station indicates that atmospheric conditions are not contributing to the observed seasonal fluctuations.

Figure 7.4: GBInSAR data illustrating total displacements (mm) between June 18 and October 17, 2008 (courtesy of Barla et al. personal communication 2009).

Figure 7.5: Leica survey data collected at target location K17 demonstrating that June-October deformation rates (absolute slope of the trend-lines) are greater than long-term trends.

Figure 7.6: Cumulative distribution curves of measured displacements for different grid densities resolved from GBInSAR raster format data.

Figure 7.7: (top) Distribution of data across the Beauregard Landslide mass, and (bottom) contoured displacement rates for data sets with varying GBInSAR data density (a) 10 m, (b) 50 m and (c) 100 m.

Figure 7.8: Comparison of slope deformation patterns interpreted using sub-sets of the Beauregard displacement data: (a) Leica only, (b) GPS only, (c)
GBInSAR only, (d) Leica and GPS, (e) Leica and GBInSAR, (f) GPS and GBInSAR.

Figure 7.9: Representation of overall slide behaviour with displacement rate standard deviation from the mean data sets with varying GBInSAR data density (a) 10 m, (b) 50 m and (c) 100 m.

Figure 8.1: (top) Beauregard landslide as viewed from the east side of the valley, with inset map showing location in northwestern Italy. (bottom) The Beauregard Dam as viewed from upstream; reservoir levels are restricted to well below design capacity due to loading by the landslide which impinges the left abutment.

Figure 8.2: Geological map of the Beauregard Landslide (geological mapping has been provided by Barla 2009)

Figure 8.3: Cross-section illustrating the location of piezometers, an inferred water table measured by a number of instruments (M17bis, PZ6, PZ7, CL3 and S1/04) and a perched water table sampled by one piezometer (PZ1). Inset shows plan view of piezometer locations and the position of the cross-section plane within the landslide boundary.

Figure 8.4: Water table interpretation using minimum curvature algorithm based on piezometric data.

Figure 8.5: Displacement rates measured by Leica, GPS and GBInSAR in mm/year (modified after Kalenchuk et al. 2010b – Chapter 7).

Figure 8.6: (top) Contoured displacement rates (bottom) representation of overall slide behaviour in terms of displacement rate standard deviations from the mean (modified after Kalenchuk et al. 2010b – Chapter 7).

Figure 8.7: Beauregard slope movements as measured at various elevations on a borehole plumb-line show correlation with ground water levels. Seasonal slope accelerations coincide with elevated water tables during summer months. Inset shows location of borehole plumb-line (modified after Miller et al. 2008).

Figure 8.8: Shear zone location has been identified in seismic sections and borehole intercepts (courtesy of Barla et al. personal communication 2008). (left) Location of seismic lines and boreholes and (right) sample seismic data illustrating location of borehole intercept, basal shear surface, and major faults.

Figure 8.9: Basal slip surface geometries interpreted using (a) a minimum curvature algorithm and (b) a multiquadratic radial basis function.
Figure 8.10: (top) Displacement rate standard deviation from the mean measured in the field vs. modelled returns $R^2$ values used to quantitatively compare various model simulations, $R^2$ values closer to 1 indicate better correlation. (bottom) Contoured displacement rate standard deviations from the global mean for used for visual comparison between modelled and measured data.

Figure 8.11: (top) Schematic illustrating base case material properties utilized in 3DEC simulations, (middle) a typical cross section through the landslide model and (bottom) isotropic view of the 3D Beauregard model looking southwest (modified after Kalenchuk et al. 2010d).

Figure 8.12: Rockmass model for the Beauregard Landslide (based on Barla et al. 2006 and laboratory testing results provided by Barla 2009).

Figure 8.13: Contour plots of displacement rate standard deviations from the mean in numerical models with varying residual frictional strength in the upper and lower regions of Beauregard Landslide (Kalenchuk et al. 2010e).

Figure 8.14: (top) Plan view of numerical models with (left) discrete landslide zones and (right) a single monolithic mass. (bottom) Contour plots of displacement rate standard deviations from the mean measured in numerical models (Kalenchuk et al. 2010e).

Figure 8.15: Hypothesized changes to ground water levels with drainage development and varying reservoir levels; (a) drainage with current reservoir levels (1700 m a.s.l.), (b) drainage with reservoir filled to 1735 m a.s.l., (c) drainage with reservoir filled to full capacity (1770 m a.s.l.) and (d) a full reservoir with no drainage.

Figure 8.16: Modelling results for changes to ground water states. (Top left) Displacement rates in mm/time step modelled under current ground water conditions. (center) Displacement rates in mm/time step modelled for varying ground water scenarios (a) drainage with current reservoir conditions, (b) drainage with reservoir at 1735 m (c) drainage with full reservoir and (d) full reservoir with no drainage system in place.

Figure A.1: Continuous three-dimensional geometries (looking northwest) interpreted using the minimum curvature algorithm with varying internal and boundary tensions and universal krigging.

Figure A.2: Continuous three-dimensional geometries (looking northwest) interpreted using radial basis functions and a moving average algorithm.
Figure A.3: Continuous three-dimensional geometries (looking northwest) interpreted using inverse distance to a power functions and linear triangulation. 345

Figure A.4: Continuous three-dimensional geometries (looking northwest) interpreted using low-order polynomials, a natural neighbour algorithm and a nearest neighbour algorithm. 346

Figure B.1: Average surface displacements achieved at a set time step interval for varying grid and mesh sizes. 408

Figure B.2: Sensitivity testing results for varying shear surface cohesion and friction values. 409
List of Tables

Table 1.1: Events which lead up to the Vaiont Landslide (summarized from Müller 1987, Semenza and Ghirotti 2000, Mantovani and Vita Finza 2003) 8

Table 2.1: Summary of spatial predictors tested for interpretation of the Downie Slide basal slip surface. 70

Table 2.2: RMSE values for a number of tested algorithms, with varying smoothing and weighting factors, assume a continuous, non-stepped, shear surface. *N:E indicates north/south:east/west. 74

Table 2.3: RMSE results for Downie Slide basal slip surface assuming discontinuous geometry at the scarp boundary. 75

Table 3.1: Summary of staged set-up for minimizing initial deformation. 112

Table 4.1: Comparison of measured movement direction to local slope aspect. *Field rate based on average between 1990 and 1999. **Slope aspect is direction of slope dip at a survey monument location averaged over a 100 m x 100 m area. 173

Table 6.1: Summary of displacement rates (mm/yr) as measured by survey monuments. 233

Table 6.2: Summary of staged set-up to minimize the initial model deformation (Kalenchuk et al 2010a – Chapter 3). 242

Table 7.1: Movement rates defined by GPS data collected November 9, 2004 and September 26, 2007. 262

Table 8.1: Laboratory testing results (results provided by Barla 2009) 295
List of Variables

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>RMSE</td>
<td>Root Mean Squared Error</td>
</tr>
<tr>
<td>n</td>
<td>Number of data points in a given data set</td>
</tr>
<tr>
<td>$Z_{estimate}^i$</td>
<td>Estimated value at data point $i$</td>
</tr>
<tr>
<td>$Z_{true}^i$</td>
<td>Measured value at data point $i$</td>
</tr>
<tr>
<td>c</td>
<td>Cohesion</td>
</tr>
<tr>
<td>$\phi$</td>
<td>Friction</td>
</tr>
<tr>
<td>$\sigma_1$</td>
<td>Major principal stress</td>
</tr>
<tr>
<td>$\sigma_3$</td>
<td>Minor principal stress</td>
</tr>
<tr>
<td>$\sigma_i$</td>
<td>Tensile strength</td>
</tr>
<tr>
<td>$\Psi$</td>
<td>Dilation angle</td>
</tr>
<tr>
<td>$E_m$</td>
<td>Rockmass modulus</td>
</tr>
<tr>
<td>$E_i$</td>
<td>Intact modulus</td>
</tr>
<tr>
<td>D</td>
<td>Disturbance Factor</td>
</tr>
<tr>
<td>GSI</td>
<td>Geological Strength Index</td>
</tr>
<tr>
<td>$\nu$</td>
<td>Poisson’s ratio</td>
</tr>
<tr>
<td>$K_n$</td>
<td>Contact normal stiffness</td>
</tr>
<tr>
<td>$K_s$</td>
<td>Contact shear stiffness</td>
</tr>
<tr>
<td>$A_c$</td>
<td>Sub-contact area</td>
</tr>
<tr>
<td>T</td>
<td>Tensile force</td>
</tr>
<tr>
<td>$T_{max}$</td>
<td>Maximum tensile force</td>
</tr>
<tr>
<td>$F_{max}^S$</td>
<td>Maximum shear force</td>
</tr>
<tr>
<td>$F^S$</td>
<td>Shear force magnitude</td>
</tr>
<tr>
<td>$F^n$</td>
<td>Normal force magnitude</td>
</tr>
<tr>
<td>$U^S$</td>
<td>Shear displacement</td>
</tr>
<tr>
<td>$U^n$</td>
<td>Normal displacement</td>
</tr>
<tr>
<td>$t$</td>
<td>Time</td>
</tr>
<tr>
<td>$V$</td>
<td>Relative velocity across a sub-contact</td>
</tr>
<tr>
<td>F</td>
<td>Hydrostatic load of the reservoir water</td>
</tr>
<tr>
<td>$\rho$</td>
<td>Density</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
</tr>
<tr>
<td>$g$</td>
<td>Gravity</td>
</tr>
<tr>
<td>$h$</td>
<td>Depth of water at inundated toe</td>
</tr>
<tr>
<td>$A$</td>
<td>Area of the landslide toe</td>
</tr>
<tr>
<td>$F_{\text{north}}, F_{\text{east}}, F_{\text{down}}$</td>
<td>Directional components of reservoir buttress load applied to numerical simulations.</td>
</tr>
<tr>
<td>$c_i$</td>
<td>Cosine value</td>
</tr>
<tr>
<td>$e_{fi}, n_{fi}$</td>
<td>The north and east components of movement measured at point $i$</td>
</tr>
<tr>
<td>$e_{mi}, n_{mi}$</td>
<td>The north and east components of movement modelled at point $i$</td>
</tr>
<tr>
<td>$J_{1, 2, 3}$</td>
<td>Joint sets at Downie Slide</td>
</tr>
<tr>
<td>$Y$</td>
<td>Piezometric elevation</td>
</tr>
<tr>
<td>$i$</td>
<td>Year</td>
</tr>
</tbody>
</table>
## List of Acronyms

<table>
<thead>
<tr>
<th>Acronym</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>3DEC</td>
<td>3-dimensional Distinct Element Code</td>
</tr>
<tr>
<td>BC Hydro</td>
<td>British Columbia Hydro and Power Authority</td>
</tr>
<tr>
<td>CVA</td>
<td>Compagnia Valdostana delle Acque</td>
</tr>
<tr>
<td>DEM</td>
<td>Digital Elevation Model</td>
</tr>
<tr>
<td>DSGSD</td>
<td>Deep Seated Gravitational Slope Deformation</td>
</tr>
<tr>
<td>ENEL</td>
<td>Italian Electric Energy Company</td>
</tr>
<tr>
<td>FLAC</td>
<td>Fast Lagrangian Analysis of Continua</td>
</tr>
<tr>
<td>GBInSAR</td>
<td>Ground-Based Interferometric Synthetic Aperture Radar</td>
</tr>
<tr>
<td>GPS</td>
<td>Global Positioning System</td>
</tr>
<tr>
<td>GSI</td>
<td>Geological Strength Index</td>
</tr>
<tr>
<td>InSAR</td>
<td>Interferometric Synthetic Aperture Radar</td>
</tr>
<tr>
<td>LiDAR</td>
<td>Light Distancing And Ranging</td>
</tr>
<tr>
<td>RMSE</td>
<td>Root Mean Squared Error</td>
</tr>
<tr>
<td>SAR</td>
<td>Synthetic Aperture Radar</td>
</tr>
</tbody>
</table>
CHAPTER 1

General Introduction

The study of landslide hazards has become increasingly important with the expansion of modern infrastructure such as pipelines and hydro-electric facilities, and societal development in response to increasing population in mountainous, landslide prone terrain. This thesis focuses on massive, active, composite landslides. Hazards associated with massive, slow-moving landslides are most often related to gradual damage of infrastructure by continuous creep. Gradual damages, while costly, pose little risk to life and are often manageable by routine maintenance. In the event of rapid acceleration, an active, slow-moving landslide can fail catastrophically. Without a reliable early warning system and functional emergency response plan, rapid failure poses much more significant consequences.

When movements are slow and a landslide is massive, the complete stoppage of movement is difficult, if not impossible, to achieve. The most reasonable approach to landslide hazard management may be to live with instability, understanding the associated risks. Hazard mitigation can then be accomplished through continuous monitoring and analysis of slope dynamics in order to recognize significant changes in landslide behaviour and to assess trigger scenarios that may initiate slope acceleration. Hazard
management is improved by knowledge of slope mechanics which can be applied to the
risk cost-benefit evaluation of engineered mitigation techniques, such as the development
of slope drainage infrastructure.

Massive landslide systems have complex deformation mechanisms with displacement
magnitudes and orientations varying, both spatially and temporally, throughout the slope.
Irregular displacements are influenced by topography, ground water, heterogeneous
rockmass characteristics, and non-uniform shear surfaces. Currently no standardized
approach has been developed for interpreting the displacements and deformation of large
landslides. Standard practices for slope stability assessments utilize two-dimensional, and
sometimes three-dimensional techniques which assume oversimplified geometries and
geological conditions. The complex and variable nature of massive slides has limited
their understanding and discouraged the development of a rigorous analysis strategy.

This thesis is motivated by the need for a multi-dimensional approach to landslide
assessment, overcoming the inadequacies of simplified models. Landslide analyses must
step away from self-simplifying two-dimensional techniques and use three-dimensional
approaches capable of accounting for complex geometry, geology, geomechanics and
hydrogeology when sufficient site specific data are available to calibrate more complex
models. A further shift from three-dimensional approaches towards four-dimensional
modelling is required to develop a knowledge base for why and how slope deformation
varies temporally.
1.1 Goals of the Thesis

This thesis focuses on improving knowledge of massive landslide geomechanics by studying how ongoing slope deformation is influenced by:

- shear surface geometry,
- spatial variation in material strength parameters,
- movement on internal shears, and
- the interaction between discrete morphological landslide zones.

The analysis of massive, active landslides starts with thorough site investigations to define the geological, geomorphological and hydrogeological settings. The importance of thorough site investigation is demonstrated by case studies of the Downie and Beauregard landslides, which are both of concern to dam and reservoir hazard management. This research aims to improve the interpretation of site conditions through the development of methodologies for making fully three-dimensional interpretations of spatial data pertaining to geometrical, geological and geomechanical landslide characteristics. Four-dimensional analyses advance understanding of temporal changes in site conditions; for instance, variable slope behaviour in response to fluctuations in ground water conditions. Both the Downie and Beauregard landslides have extensive records of slope monitoring and observational assessment. These records were used in this research for forensic analyses of spatially discriminated slope deformation and of the response of displacement rates to ground water fluctuations.

A significant theme in this thesis is to advance state-of-the-art numerical landslide modelling. A methodology for model development and calibration has been established
in order to simulate observed slope behaviour. As an extension to model development and calibration, an approach to forward testing trigger scenarios and engineered mitigation techniques is demonstrated.

1.2 A Brief History of Massive Landslides

Throughout history, a number of massive landslides have left their mark on society. Some of the most famous catastrophic events are the slides at Vaiont (Italy), Frank Slide (Canada) and Goldau (Switzerland). Several massive, slow-moving landslides have been recognized today, including La Clapière (France) and the two case studies presented in this thesis. The following sections review influential case studies which are relevant to this thesis, as they demonstrate the complexity of massive landslides, the hazards associated with them, their significant impact on society, the difficulty in analyzing slide behaviour and the uncertainty in understanding the mechanisms contributing to instability. Vaiont, Frank and Goldau were all devastating failures, providing motivation for hazard management through improved understanding of massive landslide geomechanics. Vaiont demonstrates the challenge in quantifying significant deformation rates and changes to these rates that are indicative of rapid failure. Frank and Vaiont are both cases where the numerous mechanisms contributing to failure have been identified, but the key geomechanical factors and triggers controlling the ultimate events are still widely debated. The ultimate trigger at Goldau is attributed to rapid snow melt and periods of heavy precipitation, and this case study illustrates the important temporal influences of ground water on slope deformation. La Clapière is an excellent example of strategic risk management for landslide hazards and demonstrates the value of rigorous site investigation for the enhancement of landslide technical expertise.
1.2.1 Vaiont

The Vaiont Landslide (Figure 1.1) is located in the southern Dolomites of northern Italy. On October 9, 1963, approximately 150 million m$^3$ of water were contained behind the Vaiont dam when a massive landslide, estimated to be about 275 million m$^3$, failed into the reservoir. Two lateral kilometers of the reservoir (Figure 1.2) were completely filled at heights up to 175 m above the reservoir level (Kiersch and Asce 1964). A catastrophic wave overtopped the dam washing out the valley below (Figure 1.3) and also generated upstream flooding to elevations more than 260 m above the water table. 1925 lives were lost as several villages were destroyed (Müller 1987). Remarkably the dam remains intact.

Figure 1.1: The Vaiont Landslide as viewed looking south from across the valley, inset shows location in northeastern Italy (photograph taken in May, 2008).
When the Vaiont dam was under construction, it was not standard practice to conduct upstream slope stability analysis. In 1959, a landslide failed into a nearby reservoir, resulting in a wave which overtopped the dam but did not cause any serious damage. With the Vaiont dam in late stages of construction, this incident triggered a more detailed
assessment of landslide potential in the Vaiont Valley. The new geological survey identified the outcrop of a failure surface, where bedrock transitioned to mylonite to very fractured rock, at 920 m a.s.l. (above sea level) in the basins of tributary streams to the Massalezza (Figure 1.2); this brought to light a large, ancient landslide.

Field investigation of the ancient landslide included boreholes, seismic surveys and surface displacement measurements (Semenza and Ghirotti 2000). Müller (1987) recognized the potential for re-activation, estimated that the failure volume would be 250 million m³ and hypothesized that the canyon would gradually close and that small failures would progressively drop into the reservoir, gradually developing a supporting buttressing force at the toe of the slide. With knowledge of the large slow-moving landslide it was decided that impounding of the reservoir would be carried out in steps. Daily surface monitoring of the slow creep was carried out, Table 1.1 summarizes the chronology of reservoir filling.

This catastrophe illustrates the challenges associated with interpreting slope monitoring data. When landslides are identified in the vicinity of reservoir infrastructure, risk assessment is complicated by difficulties in defining how critical the slope instability is. For instance; defining a rate of movement or a magnitude of displacement that is indicative of near or immediate mass movement is no easy task. It was accepted that the slow-moving instability would gradually fill the reservoir and the risks; possible upstream flooding and loss of reservoir capacity, associated with gradual creep were understood and accepted. However, gradual creep turned to catastrophic failure when slope acceleration occurred very suddenly. The high velocity achieved during failure was not
anticipated and in a matter of seconds the slide mass moved about 360 lateral meters and
climbed about 140 meters up the opposing bank (Müller 1987).

Table 1.1: Events which lead up to the Vaiont Landslide (summarized from Müller 1987,
Semenza and Ghirotti 2000, Mantovani and Vita Finza 2003)

<table>
<thead>
<tr>
<th>Year</th>
<th>Event</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>1960</td>
<td>March</td>
<td>Reservoir level reached 590 m a.s.l. and small displacements were observed.</td>
</tr>
<tr>
<td></td>
<td>June</td>
<td>Reservoir level reached 600 m a.s.l. and small mass movements occurred near the lake. Three boreholes were drilled to locate the ancient slide failure surface, but nothing was found; the old slide was deeper than anticipated. Geological mapping identified the bedrock-mylonite-fractured rockmass transition zone.</td>
</tr>
<tr>
<td></td>
<td>Late October</td>
<td>Movement rates accelerated to 30mm/day. A 1 m wide, 2.5 km long fissure developed at the slide crown, corresponding to the bedrock-mylonite-fractured rockmass transition mapped in June.</td>
</tr>
<tr>
<td></td>
<td>November 4</td>
<td>Reservoir level reached 650 m a.s.l., and 700,000 m³ of material failed into the reservoir (see location in Figure 1.2). A second seismic survey showed that the rockmass was more severely fractured than it had been the previous year.</td>
</tr>
<tr>
<td></td>
<td>November - December</td>
<td>Reservoir level lowered to 600 m a.s.l., displacement rates decreased to 3 mm/day.</td>
</tr>
<tr>
<td>1961</td>
<td>February</td>
<td>Müller suggested that continued and controlled lowering of the reservoir level would mitigate the displacement rates. Other mitigation suggestions included; drainage (drain adits were attempted, but work was too dangerous), removal of millions of m³ of material, cementing the mass, especially along failure surface, or building a buttress (were all considered to be impractical). Movements diminished and eventually stopped.</td>
</tr>
<tr>
<td></td>
<td>July to October</td>
<td>Four piezometers were installed, three of which operated until 1963.</td>
</tr>
<tr>
<td></td>
<td>October</td>
<td>A by-pass tunnel was completed to avoid upstream flooding should the slide block the reservoir. Reservoir filling was re-initiated.</td>
</tr>
<tr>
<td>1962</td>
<td>October</td>
<td>Reservoir level and displacements rates reached 695 m a.s.l. and 10mm/day.</td>
</tr>
<tr>
<td></td>
<td>October</td>
<td>Heavy rain occurred, and displacement rates continued to slowly increase.</td>
</tr>
<tr>
<td></td>
<td>November</td>
<td>Rain continued, and displacement rates of 12 mm/day were reached.</td>
</tr>
<tr>
<td></td>
<td>December</td>
<td>Reservoir reached 700 m a.s.l., displacements reached 15 mm/day. Reservoir lowering began again.</td>
</tr>
</tbody>
</table>
**1963**

<table>
<thead>
<tr>
<th>Month</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>March</td>
<td>Reservoir reached 650 m a.s.l., the surface movements stopped. Müller hypothesized that the displacements reflect the effect of saturation the first time, since velocities were lower during the second impounding. The reservoir level was brought up again.</td>
</tr>
<tr>
<td>May</td>
<td>Reservoir level reached 696 m a.s.l., and small displacements reached 3 mm/day (seemed to confirm Müller’s hypothesis).</td>
</tr>
<tr>
<td>July</td>
<td>Reservoir level and displacement rates reached 702 m a.s.l. and 4 mm/day.</td>
</tr>
<tr>
<td>July</td>
<td>Reservoir level and displacement rates reached 705 m a.s.l. and 5-8 mm/day.</td>
</tr>
<tr>
<td>August</td>
<td>Heavy rain fall, reservoir reached 710 m a.s.l., displacements rates increased (the western portion of the slide reached 32.5 mm/day)</td>
</tr>
<tr>
<td>September</td>
<td>Displacement rates reached 39 mm/day through the upper and lower parts of the western slope.</td>
</tr>
<tr>
<td>October</td>
<td>Reservoir level reduced to 700m a.s.l., displacement rates still increased to 2000 mm/day. Catastrophic slope failure occurred on October 9.</td>
</tr>
</tbody>
</table>

Vaiont Landslide is, without a doubt, the most studied massive landslide in the literature, and despite the many contributions by various authors, there remains considerable uncertainty in fundamental geomechanical components controlling instability, such as geology and slope geometry. Interpretation of the geological setting in the Vaiont Valley varies slightly depending on the timing, and authorship of different investigations. Figure 1.4 demonstrates a few examples of how the interpretation of Vaiont geology has evolved over time; illustrating subtle differences between different authors. The geology is primarily limestone and major formations are described as:

- **Lower-Jurassic Lias**: alternating beds of thin grey limestone and thin reddish sandy marl.

- **Mid-Jurassic Dogger**: a massive sequence of medium to thickly bedded dense grey limestone, clay parting seams are common and some breccias are found, particularly in the thin upper horizons.
- Upper-Jurassic Malm: white to reddish, platy to very thinly bedded limestone, clay seams are common, with some siliceous beds and claystone interbeds.

- Lower-Cretaceous: white, very thin to medium bedded limestone and claystone.

- Upper-Cretaceous: thin red marl beds alternating with thin, light-red limestone.

Figure 1.4: Geological interpretations of Section AA’ (See Figure 1.2 for section location) (top: Kiersch and Asce 1964, middle: Kiersch 1988, bottom: Semenza and Ghirotti 2000 after Ghirotti 1993).
The Vaiont failure highlights the paramount importance of site investigation to gain an understanding of the geological events that have shaped a region and how such events have modified structural features and rockmass properties. The geological evolution of the Vaiont region contributes to structural controls of this instability. Bedding planes are inclined, dipping into the valley on the south limb of a monocline (Mantovani and Vita-Finzi 2003). Cross-folding has tilted these beds to the east along the synclinal axis and flexure of the monocline contributes to the distinct seat-shape of the valley. The 1963 failure, as well as an ancient slide in valley have both been related to a tectonic bedding-plane fault by Hendron and Patton (1987) based on observations of shearing, slickenlines and 15 cm grooves with 1-2 m wavelength on an azimuth of 353° that predate sliding. Kiersch (1988) noted tectonic faulting throughout the area of the ancient slide and also concluded that the eastern margin of the 1963 slide area is bounded by a prominent fault.

Rockmass quality has been compromised by Pleistocene glaciation, which contributed to the formation of well developed valley-wall-parallel rebound joints, and rapid post-glacial erosion of the deep gorge gave rise to a second set of relief joints parallel to the gorge walls. Weathering of these shallow joint sets, in addition to tectonic fracturing associated with regional faulting and folding creates a very blocky, weak rockmass zone approximately 100 to 150 m thick. Below this zone the rockmass is relatively undisturbed (Kiersch and Asce 1964). Further, solution weathering of the limestone has developed sinkholes, tubes and karst features in the upper slopes (Kiersch and Asce 1964).

The lower portion of the slide was displaced laterally 360 m across the gorge, and then about 140 m of upward thrusting took place at the toe of the slide creating a 50 to 100 m
high rock countercarp sub-parallel to the canyon wall (Müller 1987). The top of the slide achieved maximum displacements up to 620 m in the west and 890 m in the east (Müller 1987). Secondary failures accommodated displacement at the sharp angle of the seat-shaped flexural fold, while the west and lower portion of the slide were displaced laterally, and the upper portion of the slide thrust over the lower, developing a large fault within the slide mass (Müller 1987).

Prior to failure, no distinct sliding surface existed in the lower-west portion of the slide, and the slip surface in this area of the slide was formed during slope failure. In the upper-west and the east portions of the slide, shear failure followed bedding planes within the Malm formation, leaving the lower portion of the Malm and the Dogger formation in place. The smooth bedding surfaces in the Malm, with estimated friction angle between 25° and 30°, dip into the valley at approximately 30° in the west and 35° in the east, shallowing to sub-horizontal near the syncline axis. Post-slide coring concluded that several layers of weak clay, up to 10 cm thick, existed along much of the 1963 slide surface (Müller 1987). The origin of these clays is thought to be sedimentary, rather than being a product of post-depositional alteration (Mantovani and Vita-Finzi 2003). The east side of the landslide is bounded by a dominant side scarp. Smaller-scale continued failure is evident by the scree slopes observed on the exposed slip surfaces.

Composite failure mechanisms contributed to the eventual catastrophic failure of the Vaiont Landslide. Hendron and Patton (1987) state that differential movements occurred due to the valley geometry and that the mass did not fail all at once; a secondary failure occurred in the east portion of the slide as support at the toe was lost during the main failure. In 1960 Müller (as summarized by Nonveiller 1987) recognized deep instability
with two zones of sliding separated by the Massalezza. The east and upper west zones featured sliding motion, while the lower west zone was creeping with distortional deformation and rotation of blocks. Given the composite and complex nature of this massive slope failure, it would be inadequate to use two-dimensional techniques or simplistic three-dimensional (particularly continuum) methods to analyze slope movements.

There is an obvious three-dimensional component to the Vaiont landslide that must be taken into account when assessing the mechanics of failure. Many two-dimensional analyses utilize a simplified seat-shape failure surface and bedding plane geology. However these overlook the fact that the Vaiont Valley is slightly concave from east to west with varying slope angles. Geometric characteristics of the failure surface vary spatially: the west is quite planar, the east appears to be more stepped and the central portion, in the Massalezza Valley shows the undulating and folded bedding plane surfaces. Hendron and Patton (1987) noted the need for three-dimensional assessment, summarizing from Guidici and Semenza (1960) that the seat portion of the failure surface actually dips upstream and taking into account the side scarp which would have carried significant shear load given the low strength clay layers along the base.

Despite the fact that this slide is so well studied, the mechanics of the Vaiont Landslide are still speculative and continue, to this day, to be highly debated. The Vaiont failure is related to a complex combination of many geological and hydrological features. Several reasonable hypotheses for the cause of Vaiont’s sudden failure have been suggested and these draw from different aspects of landslide mechanics. The following section describes a number of adverse factors that likely contributed to slope instability.
Hydrological Conditions

Multiple aquifers have been interpreted in the Vaiont Valley, separated by impermeable clay surfaces (Hendron and Patton 1987, Semenza and Ghirotti 2000). Stability of the landslide mass likely would have been most influenced by water contained in the Dogger and lower Malm formations, where recharge by surface runoff would have been facilitated by fractures and karsts, particularly at higher elevations. Since the groundwater below the slide is thought to have been related to precipitation, it is hypothesized that long rainfall would have increased the pore water pressure, reducing effective stress along the failure surface (Semenza and Ghirotti 2000). The seat-shaped folding of the limestone bedding and the impermeable nature of interbedded clay layers would have contributed to ground water confinement within and below the mass, and uplift may have been enhanced by the swelling nature of some clay minerals.

Figure 1.5 illustrates the history of precipitation, reservoir level, slope movement rates and piezometric data between 1960 and 1963. The Vaiont failure was undoubtedly contributed to by the variable reservoir levels and periods of high rainfall. Increases in slide movement roughly follow periods of increased precipitation, and correspond to high reservoir levels (Hendron and Patton 1987, Mantovani and Vita-Finzi 2003). Displacement rates increased significantly with the initial filling of the Vaiont Reservoir. Inundation of the landslide toe would have raised pore pressure conditions, generating uplift. Fractures and solution opening would have aided in the rate of water table change. Gradual, controlled lowering of reservoir levels seemed to lower displacement rates, as is evident in the reservoir chronology summarized in Table 1.1.
Figure 1.5: History of precipitation, reservoir level, displacement rates and piezometric levels between 1960 and 1963 (based on Müller 1987).

**Geological Structure and Material Strength**

The dominantly limestone bedding units dip into the valley, and are flattened near the axis of the syncline. With the exception of the load provided by the water in the reservoir there was no toe support acting on the landslide mass because the flattened portion of the bedding planes daylighted in the gorge walls (Kiersch 1964). The rockmass was highly fractured by two sets of rebound joints, bedding planes, tectonic faults and previous landslide activity. Rockmass strength was further reduced by weak interbedded clay layers and susceptibility to limestone solution weathering.
It is widely accepted that the Vaiont Landslide was predisposed to movement on a pre-existing plane of weakness. The character of this slip surface is interpreted by various authors to be influenced by the slip of an ancient landslide, weak bedding planes and/or a pre-existing tectonic fault, and these scenarios are still under debate. Given the extremely high rate of failure (25-30m/sec (Müller 1964), 25-50m/s (Jaeger 1972), 18m/s (Hsu 1969), summarized by Chowdhury 1978), it has been concluded by some that a very low friction angle would be required. Some authors support low frictional strength of the failure surface; for instance Semenza and Ghirotti (2000) recognized montmorillonitic clay along the failure surface, and also outside the slide zone, which would account for a very low friction angle (8° to 10°) and also constitute a create impermeable layers. Hendron and Patton (1987) observed multiple clay interbeds that are continuous over large areas of the sliding surface, and occur at the base of the Lower Cretaceous units. They describe these clay rich units as present in the lower portion of the slide debris and along most of the sliding surface, stating that while these units primarily conform to adjacent bedding planes, they do on occasion cut bedding planes, and resemble clayey gouge from past landslide or tectonic activity. However, other authors do not support such low frictional strengths. For instance, Broili (1967 in Müller 1987) suggested that failure occurred mainly in limestone and that clay was effectively non-existent, therefore the friction angle would not likely be less than 28° and artesian pore pressure would have been required to explain the limit-equilibrium conditions.

It is widely accepted that ancient landslides have occurred in the Vaiont Valley (Guidici and Semenza 1960 in Hendron and Patton 1987). A 1959 geological survey identified various ancient landslides, but only one was recognized as potentially dangerous on the
south side just upstream of the dam (from technical study by Müller and mapping by Semenza as summarized in Semenza and Ghirotti 2000). Kiersch (1988) mentions a large prehistoric slide along the southern margin of the inner Vaiont Valley as well as one near Casso on the north side of the valley. The extent and location of ancient slope instability is not well defined and the degree to which a prehistoric failure was a contributing factor to the 1963 instability is debated. Müller (1968 in Hendron and Patton 1987) concluded that if an ancient slide was present it would not be large enough to coincide with the actual slip surface. In contrast, Hendron and Patton (1987) state that the 1963 slide was a reactivation of an old slide based on surface morphology (for instance: remnants of old slide mass, enclosed depressions, deranged drainage, bulging slopes and other related features evident in aerial photographs).

**Dynamic Landslide Conditions**

The fractured rockmass of the steep gorge walls would have been conducive to gravitational creep. Exploration drifts in 1961 exposed clay seams and small-scale slip surfaces, and drillholes bored near the crown of the slide were slowly sheared off, confirming the progress of creep (Kiersch and Asce 1964). There is evidence for progressive detachment starting in the upper portions of the slope and progressing towards the valley (Semenza and Ghirotti 2000), and pre-failure observations of slope behaviour indicate that areas higher up on the slide tended to move faster than did lower elevations (Müller 1987). Chowdury (1978) assessed slope failure, taking into consideration the ratio between stresses parallel and perpendicular to the slope. He found that progressive failure could eventually lead to the sudden failure where pore pressures reflect reservoir conditions and frictional strengths are not unreasonably low.
Discrepancies between the very low friction values, back-calculated from high failure velocities, and frictional properties observed at different points along the failure surface may be explained by progressive failure. These discrepancies may also be explained by dynamic friction. Tika and Hutchinson (1999) demonstrated by using ring shear tests that during fast stages of failure, friction can be reduced as much as 60% below residual, and others (Semenza and Melidoro 1992 in Semenza and Ghirotti 2000) postulate that frictional heating may have developed in the slip zone during final movement; this would decrease the shear strength and accelerate displacement.

Seismicity

Some authors suggest seismicity as a contributing trigger to the Vaiont Landslide. Mantovani and Vita-Finzi (2003) hypothesized that although there is no record of seismicity at the time of the landslide, the Friuli-Venezia Giulia is known to be one of the most seismically active regions in Italy. They summarize evidence of regional seismic activity including, (from Caoli 1966, Caoli and Spada 1966 and Migani 1968) numerous micro-seismic events that were recorded between 1960 and 1963 that may have either been related to changes in the reservoir level or regional stress fields characterized by north-south compression, and (from Merlin 1997) an earthquake was felt in the region on Sept 2, 1963 with the reservoir level was at 710 m asl. If a seismic event did occur, it would have contributed to instability by shaking, and created the potential for raised pore pressure induced by shaking.

It is fairly certain that the Vaiont landslide failure was controlled by a combination of adverse geological and hydrological factors, including:

- adverse geology and hydrogeology,
• composite failure mechanisms,

• obvious three-dimensional geometry, and

• spatial and temporal variation in pre-failure observation of slope deformation.

From this, it is clear that a sophisticated approach to complex landslide analysis is required.

1.2.2 Frank Slide

On April 29, 1903 a massive landslide occurred on the east face of Turtle Mountain southwestern Alberta, Canada (Figure 1.6). The landslide is estimated to have taken 100 seconds (McConnell and Brock 1904 in Cruden and Krahn 1978) and in its path covered the entrance to Frank mine, dammed the Crowsnest River, and buried the south end of Frank and the Canadian Pacific rail line (Cruden and Krahn 1973). The debris is estimated to average 13.7 m depth over 2.7 km$^2$, with a total volume of 30 million m$^3$ (McConnell and Brock 1904 in Cruden and Krahn 1978). Frank Slide has been classified as a complex, extremely rapid, dry, rockfall-debris flow (Cruden and Varnes 1996).

Cruden and Krahn (1973) provide a comprehensive review of the Frank Slide geological setting and conditions contributing to instability. The local geology is predominantly Paleozoic limestone (Figure 1.7). The Turtle Mountain anticline contributes to bedding planes dipping unfavorably to the east; the failure surface followed these bedding planes. The narrow hinge zone located near the mountain crest features tension cracks parallel to the strike of bedding planes (Figure 1.8). The rockmass quality is compromised by three sets of discontinuities; bedding plane joints and two orthogonal families. Folding and faulting processes contributed to the shear strength of bedding planes, joints and shears.
being near residual. Further, striations related to flexural-slip folding are observed parallel to the dip of bedding surfaces. Major and minor thrust faults also have some significance; for instance the toe is controlled by minor flat-lying thrust faults.

Figure 1.6: Frank Slide as viewed today looking west, inset shows location in southwestern Alberta, Canada (photograph taken in August, 2008).

Figure 1.7: Typical cross-section through Turtle Mountain (1) Banff Formation, (2) Livingstone Formation, (3) Fernie Group, (4) Kootenay Formation, (5) Blairmore Group (Cruden and Krahn 1978).
Figure 1.8: (top) LiDAR (Light Detection And Ranging) data of the Turtle Mountain area, (bottom) close up of the mountain crest illustrating visible tension cracks (data courtesy of Froese and Moreno 2005).
The Frank Slide demonstrates the complexity of massive landslides where multiple factors contribute to composite instability including: shear failure along bedding planes, extension and failure through intact rock. There is uncertainty in the precise mechanisms and order of events contributing to the Frank Slide. The ultimate event may have initiated with progressive failure through the upper slope due to gradual creep of the rockmass under its own weight. Translation of the quasi-stable upper slope may have then loaded the lower slope until eventual and rapid rupture occurred. Alternatively, failure may have been initiated in the lower slope where coal mining provided the necessary release to effectively debuttress the upper slope. Coal mining is a widely accepted influence leading up to failure as the effects of excavations near the toe would have contributed to increased fracturing and rockmass dilation which would have promoted shearing and reduced shear strength, increased rockmass permeability and also influencing drainage (Benko and Stead 1998). Terzaghi (1950) suggested that creep of the softer rocks near the landslide base was accelerated by coal mining operations. This in turn would have contributed to loss of cohesion in the harder, jointed rocks forming the upper slopes. This process would have progressively brought the slopes nearer to limit equilibrium over time, leading up to failure. In addition to defining the key mechanisms of instability, it is also difficult to pinpoint the ultimate trigger event; possible triggers include ice-wedging at the top of the mountain, earthquake tremors, water pressures and coal mining activities.

It should be noted that monitoring and analysis of Turtle Mountain is ongoing to improve understanding of South Peak deformation (Figure 1.6) and to assess the failure potential of a second large rock avalanche (Froese et al. 2009, Brideau et al. 2009). Four zones of instability have been recognized with composite mechanisms of deformation including;
toppling, block sliding, wedge failure and subsidence (Froese et al. 2009, Brideau et al. 2009). Continued studies of Turtle Mountain are important to risk management of the South Peak as they contribute to optimization of slope monitoring practices, recognition of baseline conditions for the establishment of early warning thresholds and improved knowledge of slope geomechanics.

1.2.3 Goldau Rockslide

In September, 1806 the 40 million m$^3$ Goldau Rockslide (Figure 1.9) occurred in Switzerland, 457 lives and significant property were lost. The rockslide at first glance appears to have fairly straightforward kinematics with simple planar sliding along weak bedding units. This rockslide occurred in the sub-alpine molasse of the Swiss Alps (Thuro and Eberhardt 2003). The molasse consists of thick conglomerate units with thinner marls and sandstone interbeds, and the bedding planes dip unfavorably into the valley. A closer look brings to light more complex factors, such as the discontinuity and variable thickness of bedding units. Also, the presence of a significant side scarp highlights the three-dimensional component of this large slope failure.

Several factors have been recognized as contributing to the mechanisms of slope instability at this site. The marl units tend to weather more readily than more competent conglomerate strata (Figure 1.10), leading to increased pore volume, decalcification and reduced intact strength (Thuro and Eberhardt 2003). Progressive failure likely occurred; the gradual development and propagation of a shear surface can result in decreased cohesion and frictional strength (Thuro and Eberhardt 2003) within the slope until the ultimate mass-movement event. Rupture of the shear surface would have likely propagated through the weakest rock units, the weathered marl, or along contact
boundaries. The landslide trigger is attributed to rapid snow melt and periods of heavy precipitation in the weeks prior to the failure which would have contributed to increased pore pressures within the slope (Eisbacher and Clague 1984 in Eberhardt et al. 2005). This case study demonstrates that knowledge of ground water conditions and the influence of ground water on slope stability are fundamental to developing understanding of massive slope failure mechanisms.

Figure 1.9: (top) The Goldau Rockslide source area, (lower right) side scarp (lower left) head scarp, and (inset) location of Goldau in central Switzerland (photographs taken in May, 2008).
Figure 1.10: The strata at Goldau are made up of weak marl units overlain by more competent massive conglomerate. (bottom) Photographs showing the nature of the conglomerate and slicken lines (left) on the conglomerate contact which indicate progressive creep (photograph taken in May, 2008). Figure 1.9 illustrates the photograph source location where bedding units have been exposed by localized recent (2002) instability.

1.2.4 La Clapière Landslide

La Clapière Landslide is located near Nice in Southeastern France, on the left bank of the Tinée River (Figure 1.11). This instability is estimated to be 60 million m$^3$, extending 1.3 km between the Tenibres and Rabuons river valleys and measuring 650 vertical meters (Follaci 1987). The landslide occurs in the metamorphic and magmatic basement rock of
the Mercantour Massif (Follaci 1987), and is principally composed of plagioclasic and lenticular-banded gneiss. A competent and continuous layer of stratified meta-diorites called the Iglière bar outcrops in the gneiss unit near the center of the slope (Figure 1.12). A lower-Triassic to upper-Cretaceous sedimentary unit outcrops on the southwest bank of the river and is partially overlain by recent fluvial and glacial sediments.

Figure 1.11: Photograph of La Clapière as viewed from the south side of the valley, inset shows location in southeastern France (photograph taken in May, 2008).

Regionally, a metamorphic foliation strikes 115°E and dips 70°NE, however it has been folded by the La Clapière fold to strike 140°E and dip 10° to 30° northeast in the landslide vicinity. Folding has favorably oriented the foliation for toppling failure, a key mechanism in the compound landslide evolution (Merrien-Soukatchoff and Gunzburger 2006, Rat 1995). Numerous fractures in the valley slopes are favorable to rainfall infiltration, which influences landslide movements (Rat 1995). Rockmass fractures also contribute to drainage, as perennial springs occur at the landslide toe and temporary springs occur higher in the slope (Guglielmi et al. 2002).
The landslide features several scarps; the head scarp has two lobes joined by a horseshoe structure attributed to a fault in the middle of the landslide trending N20°E, and minor scarps occur within the middle of the landslide and below the Iglière bar. Scree slopes
are developed just below the Iglière bar on both the east and west sides of the slide (Figure 1.12). The slope is steepest (40°) below the Iglière bar, and shallower above (25°).

Slope deformation was first noticed around 1900 and by the 1930s deformation was quite visible at the top of the landslide. Gradual slope movement became more continuous in the 1970s and increased during the 1980s. The landslide has been studied since 1977 and the site has been monitored since 1982 (Follaci 1987, 1996 in Merrien-Soukatchoff and Gunzburger 2006, Fruneau et al. 1996) using inclinometers and laser ranging, as well as more recently, SAR (Synthetic Aperture Radar) images to track slope displacements for risk management and research of slope behaviour. In addition to displacement monitoring, geological and hydrogeological studies (Gunzburger and Laumonier 2002, Guglielmi et al. 2002, Cappa et al. 2003), remote sensing (Fruneau et al. 1996) and subsurface geophysical investigations (Jomard et al. 2007) have been done.

Slope behaviour has been summarized by several authors (Casson et al. 2003, Jomard et al. 2007, Follacci 1987). Ongoing deformation responds to spring snow melt and autumn rainfall events. Data shows average velocities of 10-20 mm/day with two peaks, 100 mm/day in 1987 and 50 mm/day in 1997, which correlate to meteoric conditions. The head scarp has been observed to move faster than lower portions of the slide, causing material accumulation as well as spreading of the scree slopes. The rate of movement is relatively homogenous across the landslide, and the direction of deformation averages N200°, corresponding to the local fractures; however some variation in deformation orientation is observed particularly between the regions above and below the Iglière bar.
The size of the La Clapière is gradually increasing. Work by Casson et al. (2003) revealed that the surface area increased by 12% from 1.3 km² between 1983 and 1999, the length of the landslide increased by 130±20 m and 370±20 m on the west and east sides, respectively, and the surface area of the scree slopes doubled between 1983 and 1991. Gunzburger et al. (2002) observed open fissures which indicate that the landslide is expanding to the west into an area that is just above the village of Saint-Etienne-de-Tinée. Failure was initiated as toppling (Rat 1995, Merrien-Soukatchoff and Gunzburger 2006) and current deformation appears to be dominantly controlled by fractures and the foliation. Weathering plays an important role in the landslide mechanical properties, particularly in the alteration of mica minerals to clay. A significantly weathered rockmass makes up a large portion of the upper slope (Jomard et al. 2007).

La Clapière illustrates the complex nature of massive landslides, with compound and composite failure mechanisms. Follacci (1987, summarized in Jomard et al. 2007) partitioned the landslide into three zones, separated by two sub-vertical faults trending N20°; the behaviour of these zones ranges from polyhedral slip in the northwest to rotational slip in the southeast. Jomard et al. (2007) observed that in the upper NE lobe a secondary failure is superimposed on the major one; about 5 million m³ form a shallower instability within the main landslide. This variable slope behaviour complicates the understanding of landslide mechanics and makes prediction of future deformation difficult.

This case history is an excellent example of landslide hazard management. It has been extensively studied as the instability represents a human and economic hazard because of its active scree slopes and the possibility of the formation and collapse of a landslide dam.
which could flood both the upstream and downstream areas (Follaci 1987). A significant amount of information is available including topographic, meteoric and deformation measurements, enhancing scientific study and building technical expertise. Extensive, long-term monitoring provides a basis for advanced warning of rapid slope failure. Monitoring, in addition to other mitigation strategies imposed by the French government, such as hazard mapping and regulation of land development, strengthen the management of risks posed by this natural hazard.

1.2.5 Downie Slide

The Downie Slide is located 64 km north of the Revelstoke Dam on the west bank of the Revelstoke Reservoir, in the Columbia River Valley, British Columbia, Canada (Figure 1.13). Downie Slide was first recognized in the 1950s during recognizance mapping for possible dam locations. The potential hazard to hydro-electrical operations was recognized by the British Columbia Hydro and Power Authority and they initiated an exploration program to collect geological information and ground water data. Slope monitoring began in the early 1970s to measure ground water levels and slope deformation rates. As a remediation measure a drainage system was developed between 1974 and 1982 aiming to more than offset the water table rise that was anticipated with reservoir filling. Drainage infrastructure comprises two adits totalling 2.43 km and more than 13.6 km of boreholes. Slope dewatering was effective in slowing landslide displacement rates. Reservoir filling occurred between October, 1983 and August, 1984. Over the operating life of the reservoir gradual deterioration of the drainage system has contributed to a slow rise in water table elevations, not exceeding pre-drainage levels.
This $1.5 \times 10^9 \text{m}^3$ rockslide measures 3300 m from toe to head scarp, reaches a maximum thickness of 245 m and extends 2400 m along the river valley (Enegren and Imrie 1996). Figure 1.14 illustrates the regional geological setting. In the landslide vicinity the metasedimentary sequence is made up of a number of lithological units composed primarily of thinly bedded quartzites, semipelites, psammites, calc-silicates and marbles (Brown and Psutka 1980). The metasediments are truncated on the east bank of the Columbia River Valley by the Columbia River Fault Zone. A biotite granodiorite pluton occupies the hanging wall (Brown and Psutka 1980). The landslide is highly fractured and composed of inter-layered schist, gneiss and quartzite (Imrie et al. 1991). The dominant shear zone follows a mica foliation which dips down-slope to the east. Chapters 3 and 4 provide a more detailed review of the local geological and structural setting.

![Figure 1.13: Overview photo of Downie Slide as viewed from the east side of the valley, inset shows location in southeastern British Columbia, Canada (Photograph taken in August, 2008).](image-url)
Figure 1.14: Regional stratigraphy and structure of Downie Slide (from Brown and Psutka 1980).

Morphological regions of Downie Slide, shown in Figure 1.15, have been recognized by Piteau et al. (1978) and Patton and Hodge (1975) as; (1) the head area immediately down
slope from the head scarp, (2) the central area between the head and the toe of the slide, (3) the south knob area located at the downstream toe, (4) the north knob area located at the upstream toe and (5) the active area located between the south and north knob areas. Patton and Hodge (1975) also note subordinate areas consisting of talus covered slopes along the base of the head and side scarps and a possible shallow slide located half-way down the northern boundary. These morphological regions are largely interpreted from aerial photographs and their boundaries are not precisely delineated.

Figure 1.15: Morphological regions of Downie Slide based on Patton and Hodge (1975) and Piteau et al. (1978).
This early recognition of landslide zones, and instrumentation records indicative of spatially discretized slope deformation demonstrate that Downie Slide is a composite landslide with complex failure mechanisms controlling stability. Despite this, the Downie Slide has been routinely classified simply as a rockslide and studied as a monolithic slide mass ignoring the interaction between different morphological regions (for example: Enegren 1995, Kjelland 2004). Stability assessments have primarily been completed using two-dimensional landslide cross sections with simplified geometries and singular water tables (Enegren 1995, Kjelland 2004).

This thesis (Chapters 3 to 6) provides new observations of Downie Slide from site investigation and review of all available data pertaining to geology and geomorphology. New interpretations of slide morphology and landslide zoning are presented. Displacement and ground water monitoring data has been analyzed to define spatial and temporal variation in slide behaviour.

1.2.6 Beauregard Landslide

The Beauregard Landslide (Figure 1.16) is located on the west bank of the Valgrisenche River in the Aosta Valley of northwestern Italy. This massive, active rockslide impinges on the left abutment of an arch gravity dam. Continued loading of the dam by the instability has caused gradual closure of the arch, posing a significant hazard to the dam safety and compromising reservoir operations.
The slope geology is predominantly gneiss and mica-schist of the Gran San Bernardo Series with prasinite intercalations (Barla et al. 2006). Barla et al. (2006) have delineated the valley into three morphological zones (Figure 1.17). Zone 1 is the right slope of the valley, this zone is characterized by typical glacial morphology with features such as hanging valleys and over-steepened slopes. The rockmass of the right slope is good to excellent quality. Zone 2 is the fractured rockmass of the upper portion of the left slope bounded by the mountain ridge and the main slide head scarp. Morphological features in this zone include double or multiple ridges, trenches and accumulation of debris. Barla et al. (2006) concluded that this zone is in an intermediate stage of instability development. Zone 3 is the lower active portion of the slope. The cataclastic and mylonitic rockmass of the left slope of the valley is heavily fractured by sub-vertical shears and joints that strike parallel to the valley axis.
Figure 1.17: Morphological regions identified by Barla et al. (2006), Zone 1: Glacial morphology, Zone 2: Fractured rockmass features ridges and trenches and accumulation of debris, bounded by the mountain ridge and main head scarp and is interpreted as a stable portion of the landslide. Zone 3: Active portion of the instability, inset shows DEM of landslide slope.

Slope monitoring methods include Leica total station surveys of surface targets, displacements measured along three borehole plumb-lines (since 1967), GPS (Global Positioning System) surveys, GBInSAR (Ground-Based Interferometric Synthetic Aperture Radar) and piezometers (continuous since 2003). Slope deformation has been correlated with ground water fluctuations related to reservoir filling and snow melt (Miller et al. 2008). Conventional monitoring has provided information for the behaviour of the lower region of the slope, surface displacements average 4-6 mm/year at most points, and 11-13mm/year at two points upstream of the dam (Barla et al. 2006). Newly acquired GBInSAR data has led to the identification of a main sector of motion in the
upper slopes where total displacements reached 45 mm over a four month period (Barla et al. 2010).

Barla et al. (2006) applied two-dimensional continuum techniques (using FLAC) to explore potential driving mechanisms of the instability and the interaction between the moving lower portion of the landslide (Zone 3a) and its apparently stable upper portion Zones 3b and 2). Factors of safety resolved by the shear strength reduction technique are 1.16-1.18 for the lower portion of the slope and 1.4-1.45 for the overall slope, identifying the lower region of the slope as the most susceptible to instability (Figure 1.18).

![Figure 1.18: Zones of maximum shear strain rate resolved using FLAC (Barla et al. 2006) (left) slope toe, (right) overall slope.](image)

A rigorous analysis of global slope behaviour, presented in Chapter 7, has identified heterogeneity in displacement rates measured at points across the landslide mass. This recent interpretation highlights complexity in slope behaviour and supports the need for fully three-dimensional analysis of the entire landslide mass with discretely defined zones. Chapters 8 provides a more detailed review of this case study and results from the numerical simulation of observed slope deformation patterns as well as forward testing of the effectiveness of a proposed slope drainage scheme.
1.3  **Landslide Types**

The landslide naming and classification utilized in this thesis follows the widely accepted classification scheme proposed by Cruden and Varnes (1996). Landslide forming names describe the state, distribution and style of activity. This thesis focuses on instabilities which are in an active state, meaning that they are currently moving. Active landslides may be new or reactivated instabilities. Style of activity refers to the type of landslide movement. Downie and Beauregard landslides are composite, as different types of moment are observed at different areas of the displaced masses. The dominant types of movement for both of these case studies are translational and rotational sliding, and both show morphological evidence of retrogressive behaviour. Translational sliding describes displacements occurring on a planar or undulating surface of rupture, and rotational sliding refers to a curved or concave sliding surface. Retrogressive behaviour refers to the surface of rupture extending in the direction opposite to the movement of the displaced mass.

1.4  **Numerical Modelling of Landslides**

Generally speaking there are four approaches to slope stability analyses: limit equilibrium and continuum, discontinuum and mixed continuum-discontinuum numerical methods. All four of these methods have both two- and three-dimensional capabilities.

- Limit equilibrium provides factor of safety calculations for representative slope geometries and gives consideration to shear strength, unit weight, ground water and external loading. Method of slices or composite approaches allow for the application of limit equilibrium to non-planar or multiple failure surfaces. Limit
equilibrium gives no consideration to the stress-strain state or behaviour of intact material, limiting the adequacy of this force balancing approach for application to slopes where internal strains are necessary to accommodate deformation, and deep-seated instabilities where rockmass stress-strain states become a contributing factor in slope behaviour.

• Continuum methods do not require pre-defined slip surface geometries and are often used to generate or identify critical failure surfaces. Alternatively, pre-defined shear zones can be incorporated through localized assignment of material properties and constitutive models. Continuum techniques accommodate rockmass strain so internal deformation is achieved, however they are incapable of accounting for large strains and offset at landslide boundaries.

• Discontinuum approaches utilize contact or joint elements to discretely define shear surface geometries. This approach is capable of accommodating large strains at shear zone interfaces as block separation occurs between discrete bodies. The rigid nature of bodies in pure discontinuum code does not allow for internal material deformation.

• Mixed continuum-discontinuum modelling is capable of accounting for complex slope behaviour and mechanisms, as this approach can account for intact rock strain as well as displacements on discrete surfaces. A major advance to hybrid continuum-discontinuum modelling is the ability to generate fractures while time stepping which has been a significant contribution to the simulation of progressive damage.
Numerical models are frequently used to study landslide mechanics. Literature review reveals that numerical simulation of landslides is largely focused on the initiation and kinematics of instability (for example Benko and Stead 1998, Eberhardt et al. 2002, Brideau et al. 2006, Brideau et al. 2009) or failure velocity and run-out (for example McDougall and Hungr 2004, Sitar and MacLaughlin 1997, Xiaoyu et al. 2006). Some studies also aim to develop predictive tools for future instabilities based on back analysis to gain understanding of the mechanical behaviour (for example Segalini and Giani 2004), or to assess hazard mitigation strategies (for example Tacher et al. 2005).

Relatively little work has contributed to the numerical simulation of ongoing landslide activity to study the geomechanical factors contributing to the complex nature of continuous deformation measured in slow-moving slopes. This thesis presents an advanced application of mixed continuum-discontinuum numerical methods to simulate active slow deformation in massive landslides. The interpretation of results from active slope studies can contribute to the development of early warning systems, test trigger scenarios or support the investigation and optimization of engineered mitigation techniques (for example Tacher et al. 2005, Kveldsvik et al. 2008).

In numerical modelling, users have the choice between two- and three-dimensional approaches, each with inherent advantages and disadvantages. Two-dimensional approaches offer a geometrically simpler option and are by far more commonly utilized. They have evolved significantly from simplistic slope stability analyses using limit equilibrium approaches. A major advance to landslide modelling is an approach to total rock slope failure processes (failure initiation, transport and deposition) by Stead and Coggan (2006). By using ELFEN, a finite-discrete element code with brittle fracture
generation, this approach allows dilation, fracturing and the creation of voids which all contribute to landslide kinematics. Still, one of the principal short falls, as with any two-dimensional simulation, is the inability to account for the lateral migration of material. This limitation leaves two-dimensional analyses suitable for simplistic slides with regular lateral geometries; otherwise two-dimensions are largely inadequate for most landslide scenarios. To date three-dimensional methods are rarely used. Further, there has been little or no application of these methods to research of continuously moving, slow, slope deformation processes in well developed slopes which have already achieved residual states, as is the case for Downie and Beauregard. Where three-dimensions have been utilized, models are often oversimplified using bowl-shape failure surfaces with homogeneous material properties and monolithic slide masses. These may allow lateral migration, but otherwise lack the complexity necessary for modelling spatial variation in landslide behaviour.

Numerical methods are capable of accounting for improved interpretations of irregular geometric configurations, heterogeneous geological setting, complex hydrogeology and sophisticated constitutive models, yet slope studies often focus on only one, or rarely multiple, of these factors. A principal focus of this research is a rigorous numerical modelling study looking at how the spatial complexities in behaviour are primarily controlled by the three-dimensional slip surface geometry and heterogeneity in geomechanical shear zone properties, while also being influenced by secondary shears and the mechanical interaction of discrete landslide zones. These factors are all accounted for while applying reasonable interpretations of piezometric conditions.
It is recognized that landslide models do not have to be all-encompassing; however they do have to be able to account for those main geomechanical factors controlling behaviour so that observed deformation is adequately reproduced. Several factors must be tested in order to identify which are controlling, and when multiple factors are identified as influential, models must be capable of incorporating all of them. This added rigor may increase the cost of site investigation and involve a more detailed interpretation of site conditions in order to define the full three-dimensional distribution of key modelling parameters. However, in large, complex slopes, this rigor is capable of significantly improving the understanding of slope mechanics and adds substantial value to making informed decisions for design and hazard management.

1.5 Synopsis of Findings and Major Scientific Contributions

The main objective of this research was initially to develop numerical models capable of reproducing observed slope deformation. This goal has evolved into a multifaceted study of two landslide case studies, and has lead to the development of new data interpretation techniques, resulting in a number of major scientific contributions (Section 1.5.1 to 1.5.4). The rigorous methodology developed for sophisticated numerical modelling considerably advances numerical procedures for landslide simulation (Section 1.5.5). Further, the modelling results are a contribution to the geomechanical knowledge base of not only the specific case studies in this thesis, but also to the fundamental behavioural understanding of complexities inherent to massive slopes in general (Section 1.5.6).
1.5.1 Interpretation of Spatial Data to Generate Three-Dimensional Shear Surface Geometries

Geotechnical analysis of the behaviour of massive landslides is limited when shear surface geometry is simplified by analysis of two-dimensional sections through a slide mass or a three-dimensional geometry with spherical or bowl-shaped slip surfaces. This contribution provides a semi-automated methodology for interpreting the complex geometry of sub-surface shear interfaces in three-dimensions using spatial prediction algorithms. Geometries are interpreted using surficial (landslide mapping, aerial photographs, digital elevation models), and sub-surface (borehole geology, geophysics, excavations) data pertaining to shear zone location. Multiple, and different, surface geometries can be interpreted for any given data set. Small- and large-scale geometric discrepancies result from the use of different spatial predictor techniques and geological assumptions of surface continuity, respectively. No single interpretation technique is suitable for all applications, and the most appropriate algorithm must be selected based on statistical validity, or goodness-of-fit, spatial pattern consistency and geological compatibility.

The Downie and Beauregard basal slip surface geometries are best interpreted using the minimum curvature algorithm. The minimum curvature algorithm not only performs well statistically, but it also produces geologically realistic results well suited to the folded or undulating nature of foliation. The Downie interpretation utilizes a stepped surface to provide a sub-vertical boundary at the side and head scarps. A continuous slip surface has been interpreted for Beauregard as there is no evidence to support extension of a sub-vertical face to depth near the scarp features.
1.5.2 Temporal Extrapolation of Piezometric Data

When monitoring massive landslides for an extended period of time, the spatial distribution of instruments will vary, as some are installed or decommissioned. When data is spatially interpolated (particularly when the process is semi-automated) and the distribution of instruments changes, apparent ground water fluctuations are observed at close proximity to decommissioned or installed instruments. Unless these fluctuations actually coincide with an adjustment in ground water boundary conditions, they are probably an artifact of the changing data set.

By exploring the sensitivity of ground water interpretations to instrument decommissioning and installation, this contribution has provided a data extrapolation technique in order to avoid apparent, and incorrect, artifacts of a temporally variable data set. Application of this technique to Downie Slide piezometric records has resolved spatially discriminated magnitudes of ground water changes over the development and operation of the Revelstoke Reservoir. Slope drainage development has lowered ground water levels significantly in slide areas proximal to adits and boreholes, while portions of the slope more distal to drainage infrastructure saw no significant changes. The ground water response to reservoir filling was observed by piezometers close to the inundated portion of the landslide toe, and since reservoir filling, small increases in ground water levels have been observed in regions of the landslide near drainage infrastructure due to gradual deterioration of the drainage system.
1.5.3 Definition of Morphological Zones at Downie Slide and Interpretation of Slope Behaviour

A new classification of Downie Slide has been defined by this research. Downie Slide, in its modern state, is a massive, active, composite, extremely slow-moving rockslide. In order to interpret landslide mechanics, it is necessary to recognize whether different landslide zones exhibit variable behaviour. Zones at Downie have been identified from LiDAR data, observations of surface morphology made during site visits by the author in 2008 and 2009, and through detailed analysis of slope monitoring data. As a result the slide has been discretized into regions based on morphological features and spatially discriminated slope behaviour and failure mechanisms. Slide dynamics have been interpreted for each of the individual zones, as well as a global assessment of all zones to interpret the overall modern landslide behaviour.

1.5.4 Fusion of Monitoring Data Sources and the Analysis of Beauregard Slope Deformation Patterns

Monitoring the deformation of large landslides provides valuable data for characterizing slope behaviour and improving knowledge of slope mechanics. Multiple monitoring techniques are often used on a single slope, and as such data amalgamation is necessary to develop a full picture of ongoing displacements. Data from multiple survey sources at the Beauregard Landslide, (total station Leica surveys, Global Positioning System surveys and Ground-Based Interferometric Synthetic Aperture Radar) have been amalgamated to analyze overall landslide deformation. This data fusion has resolved high deformation rates through the upper portion of the slope, moderate rates near the southern toe, slightly
slower rates through the central portion of the slope, and extremely slow movements at the north toe.

1.5.5 Development of Numerical Modelling Methodology

Literature review has demonstrated the need for sophisticated numerical methods of massive landslide simulation. Three-dimensional numerical models have been developed utilizing 3DEC (3-Dimensional Distinct Element Code) (Itasca Consulting Group, Inc. Minneapolis, Minnesota 2003) to simulate massive landslides. This contribution provides a methodology for the application of mixed continuum-discontinuum numerical methods to represent the landslide and undisturbed in situ material using deformable continuum blocks which interact along shear zones defined by discrete discontinuities. This approach to sophisticated modelling accounts for aspects such as; mesh density and grid size sensitivity testing, the inclusion of water tables, the buttressing load imposed by a reservoir, as well as boundary conditions and in situ stresses. Rigorous model calibration has encompassed several factors contributing to the complex nature of large instabilities:

- A primary factor influencing landslide deformation patterns is the three-dimensional geometry of basal slip surfaces, to reproduce observed slope behaviour it is necessary to simulate fully three-dimensional, geologically realistic, shear surface geometries.

- Spatial variation in shear zone strength parameters should be accounted for when data is available. Heterogeneous shear zone strength parameters contribute to localized stress concentrations and joint slip; adding to intricate patterns of three-dimensional slope deformation. The distribution of parameters assigned to
models which closely reproduce measured slope deformation may be taken as the truest three-dimensional interpretation of shear surface character.

- The influence of secondary shears should be tested on a case-by-case basis as the role of internal structure varies for specific case studies. Where justified by morphological zoning it is ideal to simulate secondary shears, alternatively, in the absence of discretely defined secondary slip surfaces, a low deformation modulus can achieve reasonable results by allowing internal strains to be adequately accommodated.

- The discrete definition of morphological regions improves slope simulation by allowing for mechanical interaction between regions.

This approach to mixed continuum-discontinuum landslide modelling reproduces global deformation patterns and improves understanding of key geomechanical factors controlling massive instabilities.

1.5.6 Advances to Landslide Geomechanics by Numerical Simulation of Observed Slide Behaviour

Optimized three-dimensional numerical models have been developed to simulate, observe and compare landslide behaviour. Modelling of the Downie Slide has improved understanding of the landslide geomechanical characteristics. It has been found that: the basal slip surface geometry is best interpreted using a stepped minimum curvature algorithm, variability in shear zone thickness is best numerically accounted for by spatial distribution of contact element shear and normal stiffness magnitude, secondary internal
shears make minor contributions to global landslide behaviour, and important mechanical interactions occur between landslide morphological zones.

Through numerical studies of Beauregard Landslide it has been concluded that deformation patterns are best simulated when the basal shear surface geometry is interpreted using a continuous minimum curvature algorithm, the frictional strength of shear surfaces varies spatially and observed slope behaviour is best reproduced when the post-peak frictional strengths are 19° and 25° for the upper and lower regions of the landslide respectively. Further, differential movements are best simulated by allowing landslide zones to displace independent of one another, rather than simulating the entire slope as a monolithic mass

1.6 Thesis Format

This thesis has been prepared in manuscript style in accordance with the guidelines established by the School of Graduate Studies at Queen’s University. Chapter 2 is a manuscript that has been published and Chapters 3-8 are manuscripts that have been submitted for publication. This research has developed a rigorous process for the study of massive landslide geomechanics involving: (1) characterization of site geology, geomorphology and hydrogeology, (2) analyses of landslide monitoring data and the interpretation of modern landslide behaviour, (3) the development and calibration of numerical models capable of simulating observed slope deformation, as well as (4) testing the influence of ground water levels on temporal changes in slope behaviour.

Chapters 2-6 are papers focusing on Downie Slide; each paper centers on a different component of the case study research. Being that these papers focus on the same case
history there is some overlap in the description of site characterization and numerical procedures. Chapter 2 describes the semi-automated statistical methodology for the three-dimensional interpretation of spatial data. The main benefit of this approach over traditional hand contouring is its repeatability. This method for spatial data interpretation is applied throughout this thesis not only to shear surface geometry, but also to the interpretation of shear zone thickness and ground water levels and slide deformation rates across the full extent of a landslide.

Downie Slide numerical modelling has been divided into three components, and each of these components is tied to a specific area of site characterization. As such, Chapters 3, 4 and 6 each focus on these specific areas of site characterization and successively build on the complexity of numerical models and site analysis. Chapter 3 provides a detailed description of model development and numerical procedures. This chapter focuses on how spatially discriminated slope deformation is controlled by shear surface geometry and spatial variation in shear zone material properties, specifically stiffness as a function of shear zone thickness.

Chapter 4 focuses on the analysis of Downie Slide as a composite landslide where the slope has been discretized into a number of zones based on morphological observations and an analysis of spatial variation in slope deformation rates and failure mechanisms. This paper provides the most detailed review of the Downie Slide geological, structural and morphological setting; it also provides an analysis of slope behaviour. The numerical modelling in Chapter 4 assesses the influence of secondary shears and the interaction of discrete landslides zones on overall slide behaviour.
Chapters 5 and 6 focus on Downie Slide ground water. Chapter 5 describes a methodology which accounts for sensitivity to spatial and temporal variations in data density when interpreting ground water levels. Using the analysis results from Chapter 5, water tables representative of different phases in reservoir operation are applied to calibrated numerical models in order to test how ground water fluctuations influence simulated slope behaviour. Chapter 6 not only tests observed ground water fluctuations, it also considers possible trigger scenarios including rapid reservoir drawdown and drainage system failure.

To demonstrate the transferability of those scientific methods developed for Downie Slide a second field site, the Beauregard Landslide, has been studied and numerically simulated. Chapter 7 and 8 are papers regarding to Beauregard. Chapter 7 gives an analysis of slide monitoring data for the interpretation of modern slope behaviour. Chapter 8 provides a general overview of the site characterization, results from the calibration of numerical models, as well as numerical testing of the effectiveness of a proposed drainage system.

Chapter 9 provides a research discussion and Chapter 10 summarizes the principal findings and scientific contributions, including a complete list of publications.

1.7 References


CHAPTER 2

Application of spatial prediction techniques to defining three-dimensional landslide shear surface geometry

2.1 Abstract

This study explores the application of interpolating and non-interpolating spatial prediction algorithms to interpreting shear surface geometries. A number of spatial prediction techniques have been tested, and the most appropriate algorithms for the Downie Slide data set have been selected based on the root mean squared error (RMSE) determined from cross-validation. Visual assessment of reasonable spatial patterns has allowed for final selection of algorithms that produce geologically realistic results. Through this process the performance of a number of interpolation algorithms have been tested in terms of accuracy and the development of reasonable spatial patterns. The goal of this study has been; (a) to develop a methodology for interpolating three-dimensional

* This Chapter has been published as:
shear surface geometries, and (b) to assess which interpolation methods are most appropriate for the interpretation of the Downie Slide basal slip surface geometry, based quantitatively on RMSE and qualitatively on the geological “trueness” of the geometric output.

2.2 Introduction

The stability analysis of massive active landslides and large slope failures commonly use two-dimensional cross-sectional analyses or basic three-dimensional geometries. These simplified interpretations of the shear surface geometry do not adequately represent the true shear surface conditions in terms of shear zone location, thickness and shear strength parameters, making it nearly impossible to calibrate a three-dimensional analysis with respect to spatially distributed slope deformation measurements.

Recent improvements to slope stability analysis methods allow analysis of more complex shear surface geometries through three-dimensional numerical modelling. This has given rise to the need to develop a methodology for interpreting the true three-dimensional geometry of slip surfaces within large slopes. Interpreting sub-surface geometries is difficult due to the inherent data limitations; without expensive and extensive excavation or drilling programs, the exact location of slip surfaces within a massive landslide is usually only defined by a sparse borehole drilling program and the location of the slip surface outcrop at the landslide boundary.

2.3 Site and Data Description

Downie Slide is a massive $1.5 \times 10^9$ m$^3$ rockslide located 64 km north of the Revelstoke Dam on the west bank of the Revelstoke Reservoir, in the Columbia River Valley, British
Columbia, Canada (Figure 2.1). The slide was first recognized in 1956 during reconnaissance mapping for a possible dam site. British Columbia Hydro and Power Authority (hereafter referred to as BC Hydro), recognizing the potential hazard to future hydro-electrical developments created by slides, initiated an intensive exploration program. The program commenced in 1973 and involved collecting ground water data, monitoring slope movements, locating sub-surface shear zones by drilling and seismic methods, and undertaking remedial measures in the form of drainage tunnels (Moore et al. 1997).

Interpretation of the Downie Slide shear surface geometry is based on a number of data types. Surface mapping (Brown and Psutka 1978), aerial photographs (Department of Energy, Mines and Resources 1977) and a digital elevation model (Base Mapping and Geomatic Services 2003) identify the surface expression of morphological features and define the boundary of the landslide mass, which has been digitally resolved to 162 coordinate points. Sub-surface data is available from borehole logs, core photographs and inclinometer data; there are 42 sub-surface data points at Downie Slide. Sub-surface data collection locations are spaced at 6 to 910 meters and average 157 meters; this is equivalent to 5.9 samples per km². The boreholes are irregularly spaced; being clustered near the toe of the slope and sparse at higher elevations, due to difficult access conditions for drilling equipment, and the focus on higher rates of movement near the slide toe. Figure 2.2 illustrates the spatial distribution of sample data.

The geological setting and character of the landslide must be considered when interpreting the shear surface geometry. An understanding of the geological nature of slope instability is important in order to recognize if the interpretation results are
reasonable. For example, if a slip surface follows a pre-existing fault or bedding planes, a fairly planar geometric result may be expected for the shear surface. In a highly fractured or weak rockmass, a circular failure surface may develop, and in jointed rockmass the failure surface may be a complex system of wedges.

Figure 2.1: Location map of Downie Slide located in southeastern British Columbia, Canada.
The rockmass at Downie Slide (Figure 2.3) is highly fractured and composed of interlayered schists, gneisses and quartzites (Imrie et al. 1991). The basal shear zone occurs along micaceous layers within a weak pelitic horizon; mica foliation dips 20° down-slope towards the east, facilitating instability. It is reasonable to expect an undulating slip surface, because the regional foliation which is associated with first and second phase folding has been overprinted by a third phase of folding.
Figure 2.3: (top) View of Downie Slide looking west, (lower left) the headscarp, (lower right) folding of the interlayered schists, gneisses and quartzites observed in drainage adit tunnel located in central lower portion of the slide.

2.4 Identifying Sub-Surface Shear Location

Sub-surface conditions, such as the shear surface location, are interpreted from borehole logging and core photographs from drilling campaigns conducted by BC Hydro in 1973-1977, 1980-81 and 1992-93, as well as from inclinometer records. Geological logging defines sub-surface locations where boreholes intercept shear surfaces, and also can indicate the thickness of zones of fractured and sheared rock. Figure 2.4 illustrates a borehole log, where BC Hydro geologists have reported a shear zone within Downie
Slide. The same shear is evident in core photographs which demonstrate increased fracture frequency and zones of rubble and gouge.

Figure 2.4: Geological logging records and core photographs for drillhole S8 provided by BC Hydro demonstrate where boreholes intersect shear surfaces, and also indicate shear zone thickness. The shear zone is evident in core photos by increased fracture frequency and zones of rubble and gouge between the 645 and 708 ft. markers. Core runs are 1.5 m long.

Inclinometers identify shear zone locations by measuring sub-surface ground displacements in active movement zones. Inclinometer records can be compared to
geological borehole logs to confirm the location of shear zones, and in some cases to
identify slip-surfaces that are not obvious in geological records. This is demonstrated in
Figure 2.5. Inclinometer data also describes the behaviour of shear zones, for instance shear zones may be quite active (shears A1, A3 and B2), or may have very little
associated displacements (shear A2 and B1).

Figure 2.5: Inclinometer data for two boreholes demonstrating a shear zone (B2) that is
noted in the geological data logs, with a very obvious displacement horizon in the
inclinometer profile, shear zones (A2 and B1) that are noted in the geological data logs,
but are not apparent in the inclinometer data, and potential shears (A1 and A3) that are
apparent in the inclinometer data but not noted in the geological data logs.

Spatial prediction techniques for interpreting slip surface geometry require that known shear locations be defined by point coordinates. The borehole intercepts with the Downie Slide shear zone show a considerable range in thickness from less than 2 m to nearly 50 m. As demonstrated by the inclinometer profile in Figure 2.5, strain may occur along a discrete surface within the shear zone, or may be distributed across the shear profile. To
resolve the shear zone to a point location for the model, the mid-point of the zone is used; unless a specific location is otherwise justified by inclinometer data.

2.5 Geological Assumptions

Interpreting shear geometry requires assumptions of the continuity between the subsurface shear zone locations and the outcrop boundary of the landslide. Assuming a continuous surface between borehole intercepts and the mapped location of the slip surface outcrop interprets the shear zone to be uninterrupted; the surface may be smooth or undulating however it is not stepped or kinked. Assuming a discontinuous surface allows the shear zone to be stepped.

Shear surfaces that may be predicted based on continuous and discontinuous assumptions are schematically illustrated in Figure 2.6. The assumption of a continuous surface simplifies the interpolation of three-dimensional geometries, however, may not be geologically correct. Aerial photographs reveal a discrete scarp on the southern and western margins of Downie Slide (Figures 2.2 and 2.3); this indicates that the shear surface may be stepped in sections rather than smoothly curved between the outcrop location and the borehole intercepts. The existence of a discrete scarp may be accounted for by assuming a discontinuous shear surface. Stepped surfaces complicate the geometric interpretation of shear geometry because without exploratory drilling, the depth of the sub-vertical face extending below the topographic surface is unknown. It is important to test the sensitivity of slope stability analysis to assumptions of continuous or discontinuous surfaces. The simplified continuous surface may be less geologically correct, however if the slope stability analysis is not sensitive to these variations in shear
surface geometry the extra rigor of developing a more complex discontinuous surface may not be required.

![Figure 2.6: Assumptions of a smooth, continuous shear surface (top) and a stepped, discontinuous shear surface (bottom).](image)

**2.6 Spatial Predictors**

Spatial prediction techniques are commonly applied to geo-sciences, for instance; to generate digital elevation models, topographic models (Carrara et al. 1997, Desmet 1997, Wood and Fisher 1993), hydrological models (Borga and Vizzaccaro 1997), to map soil properties (Cambardella and Karlen 1999, Laslett et al. 1987), in environmental and contaminant mapping (Weber and Englund 1992), in climate studies (Declercq 1996) and for mining geostatistics and ore reserve estimation (Journel and Huijbregts 1978).

Spatial prediction algorithms are also suitable for evaluating shear geometry, because over short distances, sample points on geological surfaces are typically spatially
correlated, and as distance increases locations become increasingly independent (Davis 2002). Different interpolation algorithms will yield different surface geometries and there is no formal statistical theory to predict which algorithms will produce the best results (Davis 2002). Despite extensive comparison of different predictors used to determine which techniques are most suitably applied to specific data sets (for example: Declercq 1996, Brus et al. 1996, Webber and Englund 1992, 1994, Laslett et al. 1987, Voltz and Webster 1990, Carrara et al. 1997), no characterization scheme has been defined to determine the best algorithm prior to interpretation. The appropriateness of a given spatial predictor is dependent on the nature of the data set and data set characteristics such as sample spacing and distribution. Therefore it is good practice to test a number of prediction techniques in order to select the most appropriate for each specific application.

Several interpolation algorithms have been tested (Table 2.1) using SURFER software version 8.05 (Golden Software, Inc. Golden, Co., USA 2004) to determine which are most appropriate for application to the Downie Slide data set. There are two common approaches to assessing the quality of an interpolated surface, first by statistical comparison where calculated values are evaluated relative to actual values (for example Declercq 1996, Aguilar et al. 2005, Brus et al. 1996), and second by the visual inspection of spatial patterns (for example Declercq 1996, Desmet 1997, Wood and Fisher 1993, Carrara et al. 1997). Those tested for this study include: inverse distance to a power, kriging, minimum curvature, natural neighbour, nearest neighbour, a number of radial basis functions, linear triangulation, moving average and low-order polynomials (Table 2.1).
<table>
<thead>
<tr>
<th>Spatial Prediction Technique</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum Curvature</td>
<td>Interpolation using the finite difference approach to solve a minimum curvature algorithm (Golden Software, Inc. 2002), which is analogous to approximating the shape taken by a thin plate flexed to pass through data points. User-defined input parameters include internal and boundary tension, which can range in value between 0 and 1. As the internal tension approaches 1 the surface curvature increases in close proximity to sample points. As the value of boundary tension approaches 1 the solution to the minimum curvature algorithm forces the edges of the interpolated region to be flatter; when boundary tension is set to 0 the solution allows free-edge conditions (Smith and Wessel 1990).</td>
</tr>
<tr>
<td>Kriging</td>
<td>An estimation procedure that uses a variogram model for interpolation (Cressie 1990, Davies 2002). The variogram model describes how the value of a specific variable, such as slip surface elevation, varies relative to distance and direction between sample points. Several forms of kriging do exist, the most common three are; simple kriging, where the mean value is known prior to calculations and is constant; ordinary kriging, where the mean is assumed constant, but is unknown; and universal kriging, where the mean is unknown and not constant. Performance is controlled by how well the data set is fit to the variogram model. Care must be taken when applying kriging to sparse data sets because the information may be insufficient to estimate variance over short distances (Brus et al. 1996). At least 100 data points are required to produce a variogram with reasonable confidence (300 to 400 data for anisotropic variation) (Voltz and Webster 1990).</td>
</tr>
<tr>
<td>Radial Basis Functions</td>
<td>Radial basis interpolation is similar to kriging, where multivariant functions are used rather than a variogram model (Lazzaro and Montefusco 2002, Aguilar et al. 2005). Inverse Multiquadratic $B(h) = \frac{1}{\sqrt{h^2 + R^2}}$ Multilog $B(h) = \log(h^2 + R^2)$ Multiquadratic $B(h) = \sqrt{h^2 + R^2}$ Natural Cubic Spline $B(h) = (h^2 + R^2)^{1/3}$ Thin Plate Spline. $B(h) = (h^2 + R^2)\log(h^2 + R^2)$ $R^2$ is a smoothing factor which depends on the number and spatial distribution of sample points. There is no universally accepted method for selecting an $R^2$ value (Aguilar et al. 2005). In this application the $R^2$ testing has been completed for the range between 1 and 0.5 times the average sample spacing (Golden Software, Inc. 2002). Performance is sensitive to specific data sets, splines have been demonstrated to work well for surfaces with smooth slope transitions Aguilar et al. (2005), and perform poorly when sharp changes occur over short distances, such as steep cliffs. Highly undulating surfaces may occur in regions with irregularly scattered data (Declercq 1996 and Voltz and Webster 1990).</td>
</tr>
<tr>
<td>Inverse Distance</td>
<td>Weights data such that samples further away from an interpolation point have less influence than samples which are closer (Golden Software, Inc., 2002) Accuracy depends on the type of data; the type of neighbourhood search and the weighting function (Declercq 1996). Predicted values are confined within the minimum and the maximum of the sample values</td>
</tr>
</tbody>
</table>
Low-order Polynomials

Regression is used to fit a polynomial to the data set (Draper and Smith 1981).

Simple Planar
\[ z(x, y) = A + Bx + Cy \]

Bi-linear Saddle
\[ z(x, y) = A + Bx + Cy + Dxy \]

Quadratic Surface
\[ z(x, y) = A + Bx + Cy + Dx^2 + Exy + Fy^2 \]

Cubic Surface
\[ z(x, y) = A + Bx + Cy + Dx^2 + Exy + Fy^2 + Gx^3 + Hx^2y + Ixy^2 + Jy^3 \]

Low-order polynomials are not accurate for abruptly changing data; they tend to perform better for gradually changing data (Declercq 1996).

Natural Neighbour

Thiessen Polygons are distributed around sample points. Unsampled points “borrow” area from neighbouring samples which are then weighted proportion to “borrowed” area (Sibson 1981). Predicted values are confined within the minimum and the maximum of the sample values.

Nearest Neighbour

Unsampled locations are assigned the value of the nearest sample point. Predicted values are confined within the minimum and the maximum of the sample values.

Moving Average

Estimates the average value of sample points within a user defined search radius (Golden Software, Inc. 2002). The search radius is user defined, and must be less than the maximum sample spacing. Predicted values are confined within the minimum and the maximum of the sample values. Performs poorly when applied to irregularly spaced data.

Linear Triangulation

First-order planar surfaces are fit through all sets of three neighbouring sample points. Predicted values are confined within the minimum and the maximum of the sample values.

The performance of individual algorithms is influenced by the input parameters, such as smoothing and weighting factors that are assigned to control each interpolation function, therefore, user defined input were tested for individual algorithms. The search procedure for identifying neighbours also influences interpolation results. Given the small sample size for this study global sampling was used, wherein all sample points are used to estimate values at unsampled locations. Isotropic and anisotropic neighbour weighting were tested. Anisotropic neighbour weighting weights samples according to their oriented spatial proximity to an unsampled point in space. For example if a 2:1
anisotropy for north/south:east/west was applied to two data points equidistant from an unsampled location, one to the north and one to the east, the sample to the north would be given twice as much weight as the sample to the east. 2:1 anisotropy was tested for the north/south:east/west and vice-versa. These orientations were selected based on the west-dipping landslide orientation.

2.7 Statistical Comparison

The process of testing model suitability by omitting some of the sample values at a number of locations, performing interpolation, and assessing the difference between predicted and real values is common practice (Declercq, 1996). Cross-validation has been used to narrow down the number of appropriate interpolators for Downie Slide by removing one sample at a time from the set and determining the interpolated value for each point based on the remaining data points. This returns a series of error values defined as the difference between the true measured elevation value and the interpolated elevation value at each sample location.

Cross-validation does not confirm which particular interpolators are best; it is simply a means to qualify which algorithms are well suited to the phenomenon depicted in the available data (Davis, 1987). Voltz and Webster (1990) discuss some shortfalls of cross-validation, pointing out that, for instance, when kriging is cross-validated, a new variogram should be fit when each individual sample is removed. This is a time-consuming procedure. Voltz and Webster (1990) go on to suggest a “true-validation” technique which uses a completely separate, second data set. The true values of this second set are compared to predicted values that are estimated from true values of the first set. This true-validation technique may improve on standard cross-validation.
procedure, however, given the data limitations and small data set size associated with locating shear zones within large landslides, the acquisition of a second data set in this application is difficult, if not impossible.

A few techniques are commonly used to evaluate the goodness of fit of spatial predictors based on the differences between true and predicted values, such as finding the mean and median of the squared differences (Declercq 1996), or as suggested by Li (1988) the mean absolute difference between the interpolation and the true values, and the standard deviation of these differences. In this application the RMSE (root mean squared error), which is commonly used as a basis of comparison for spatial prediction algorithms, was calculated (equation 2.1) from the cross-validation output for each algorithm,

\[
RMSE = \sqrt{\frac{1}{n} \sum_{i=1}^{n} \left( Z_{\text{estimate}} - Z_{\text{true}} \right)^2}
\]  

(2.1)

where \( n \) is the total number of data points and \( Z \) is the value of data point \( i \). RMSE values are summarized in Table 2.2 for a number of algorithms that interpolated a continuous, non-stepped, surface for the Downie Slide data set. Those algorithms with the lower RMSE values have better goodness of fit.

When a discontinuous, or stepped, shear surface is assumed for Downie Slide, a fault line is imposed on the data corresponding to the scarp location on the south and west landslide boundaries. A fault line in SURFER inserts a break in the predicted surface where points in space are interpolated using only data points which are located on the same side of the fault line and a discontinuity, or step, is inserted at the fault line location to link opposing sides with a vertical face. The selection of a spatial prediction routine is limited for discontinuous surfaces in SURFER, as fault lines are not available in all algorithms.
Table 2.3 summarizes RMSE results for spatial interpolators assuming a discontinuous, stepped surface.

Table 2.2: RMSE values for a number of tested algorithms, with varying smoothing and weighting factors, assume a continuous, non-stepped, shear surface. *N:E indicates north/south:east/west.

<table>
<thead>
<tr>
<th>Method</th>
<th>RMSE</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Isotropic Weighting</td>
</tr>
<tr>
<td></td>
<td>2:1 (N:E)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Method</th>
<th>Internal tension</th>
<th>Boundary Tension</th>
<th>RMSE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum Curvature</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>16.2</td>
</tr>
<tr>
<td>0.5</td>
<td>0</td>
<td>0</td>
<td>58.9</td>
</tr>
<tr>
<td>1</td>
<td>0</td>
<td>0</td>
<td>23.8</td>
</tr>
<tr>
<td>0</td>
<td>0.5</td>
<td>0</td>
<td>27.4</td>
</tr>
<tr>
<td>0.5</td>
<td>0.5</td>
<td>0</td>
<td>31.1</td>
</tr>
<tr>
<td>1</td>
<td>0.5</td>
<td>0</td>
<td>27.2</td>
</tr>
<tr>
<td>0</td>
<td>1</td>
<td>0</td>
<td>27.0</td>
</tr>
<tr>
<td>0.5</td>
<td>1</td>
<td>0</td>
<td>29.9</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>0</td>
<td>27.2</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Method</th>
<th>Type</th>
<th>Smoothing Factor (R²)</th>
<th>RMSE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kriging</td>
<td>Universal (linear drift)</td>
<td>110</td>
<td>17.6</td>
</tr>
<tr>
<td></td>
<td>Universal (quadratic drift)</td>
<td>55</td>
<td>17.5</td>
</tr>
<tr>
<td></td>
<td>Ordinary</td>
<td>110</td>
<td>17.2</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Method</th>
<th>Type</th>
<th>Smoothing Factor (R²)</th>
<th>RMSE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Radial Basis Functions</td>
<td>Thin Plate Spline</td>
<td>110</td>
<td>18.5</td>
</tr>
<tr>
<td></td>
<td>Thin Plate Spline</td>
<td>55</td>
<td>18.4</td>
</tr>
<tr>
<td></td>
<td>Natural Cubic Spline</td>
<td>110</td>
<td>24.2</td>
</tr>
<tr>
<td></td>
<td>Natural Cubic Spline</td>
<td>55</td>
<td>24.0</td>
</tr>
<tr>
<td></td>
<td>Multiquadratic</td>
<td>110</td>
<td>18.425</td>
</tr>
<tr>
<td></td>
<td>Multiquadratic</td>
<td>55</td>
<td>18.433</td>
</tr>
<tr>
<td></td>
<td>Multiquadratic</td>
<td>27</td>
<td>18.443</td>
</tr>
<tr>
<td></td>
<td>Multilog</td>
<td>110</td>
<td>26.5</td>
</tr>
<tr>
<td></td>
<td>Multilog</td>
<td>55</td>
<td>27.5</td>
</tr>
<tr>
<td></td>
<td>Inverse Multiquad</td>
<td>110</td>
<td>159.4</td>
</tr>
<tr>
<td></td>
<td>Inverse Multiquad</td>
<td>55</td>
<td>190.2</td>
</tr>
</tbody>
</table>

74
<table>
<thead>
<tr>
<th>Method</th>
<th>RMSE</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Isotropic Weighting</td>
</tr>
<tr>
<td></td>
<td>2:1 (N:E)</td>
</tr>
<tr>
<td><strong>Inverse Distance</strong></td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>15</td>
</tr>
<tr>
<td><strong>Low Order Polynomials</strong></td>
<td>Simple Planar</td>
</tr>
<tr>
<td></td>
<td>Bilinear Saddle</td>
</tr>
<tr>
<td></td>
<td>Cubic Surface</td>
</tr>
<tr>
<td></td>
<td>Quadratic Surface</td>
</tr>
<tr>
<td><strong>Natural Neighbor</strong></td>
<td>20.2</td>
</tr>
<tr>
<td><strong>Nearest Neighbor</strong></td>
<td>36.4</td>
</tr>
<tr>
<td><strong>Moving Average</strong></td>
<td>225.9</td>
</tr>
<tr>
<td><strong>Linear Triangulation</strong></td>
<td>20.5</td>
</tr>
</tbody>
</table>

Table 2.3: RMSE results for Downie Slide basal slip surface assuming discontinuous geometry at the scarp boundary.
2.8 Visualization of Spatial Patterns

Interpolation methods with comparable accuracy, as measured by cross-validation analyses, can generate very different spatial patterns. Visual evaluation is required to determine how well spatial prediction techniques represent local and regional patterns in the data. Knowledge of the geological setting is particularly important in order to visually interpret the reliability of spatial patterns, or the degree to which the shape of the predicted surface reflects the geological nature of the shear. At Downie Slide, shears are associated with foliation (Imrie et al. 1991) which is axial planar to second phase folding (Brown and Psutka 1978). The foliation is nearly parallel to the topographic slope, dipping 20° east. Minor folds and gentle warping (Figure 2.3) are not uncommon in the vicinity (Piteau et al. 1978) so it is reasonable to accept shear surface geometry with some degree of undulation. Figure 2.7 illustrates a number of predicted geometries for the spatial predictors that returned the lowest RMSE values, as well as an elliptical parabola generated using the cubic low-order polynomial to allow comparison of a simplified spoon-shaped slip surface to more geologically realistic results. Predicted geometries for continuous shear surfaces with less favorable RMSE values are illustrated in Appendix A.
Figure 2.7: Geometries (looking northwest) and contour plots of the interpolated lower shear at Downie Slide illustrating the error values at sub-surface data points returned from cross-validation for (1) continuous surfaces (a) minimum curvature (internal tension = 0, boundary tension = 1), (b) kriging of a variogram model, (c) the multiquadratic radial basis function, (d) the thin-plate-spline function, (2) discontinuous surface (e) minimum curvature, and (3) simplified surface (f) the elliptical parabola.
2.9 Results

The shear surface geometry at Downie Slide has been interpolated from 46 borehole point locations and the mapped location of the landslide boundary using a number of different algorithms. Algorithms were tested with varying input parameters, including smoothing and weighting factors, using both isotropic and anisotropic search procedures. Cross-validation was completed for each algorithm by removing one borehole sample at a time and interpolating the expected value at that point using all remaining data. A considerable range of RMSE values (16.2 m to 1352 m) has been calculated from cross-validation results. Assessment of the reliability of spatial patterns has also been taken into consideration. The following section discusses and compares the performance of all algorithms when applied to the Downie Slide data set.

The minimum curvature algorithm performed well. The results for this algorithm are primarily controlled by the internal tension, with little influence of the boundary tension parameter. Increasing internal tension created sharp valleys at data points and increased the RMSE value. The best RMSE results were achieved by minimizing internal tension and maximizing boundary tension with isotropic neighbour weighting. Anisotropic neighbour weighting returned slightly higher RMSE values. The minimum curvature algorithm produced geologically reasonable results based on visual assessment of the spatial patterns.

Kriging also performed well. Both universal and ordinary kriging achieved comparable RMSE results and similar spatial patterns. Kriging produces geologically reasonable results based on visual assessment of the spatial patterns.
Of the radial basis functions, the natural cubic spline and multilog did not perform as well as the multiquadratic and thin-plate spline functions in terms of RMSE values. A smooth, gently undulating surface was created using the multiquadratic function which appears geologically realistic, while geometries of the other radial basis functions are not realistic. The natural cubic spline and the thin-plate spline created excessive peaks and valleys near data points and the multilog function created a very smooth surface with distinct, sharp valleys at the sample locations. The inverse multiquadratic returned very high RMSE values and generated a near planar surface with extreme peaks and valleys at sample locations.

The performance of the inverse distance method depends on the value of the power applied. Inverse distance to the power of 10 minimized the cross-validation error for this interpolator, however, still only performs moderately well compared to the number of alternate algorithms which scored lower RMSE values. As the power value increases from 1 to 15, the resulting surface evolves from one that is fairly planar with sharp valleys at sample locations to one that closely resembles a step-like polygonal surface.

The low-order polynomials; planar, bilinear saddle, cubic surface and quadratic surface, all performed poorly. These algorithms generate very basic geometries, similar to those used in simplified slope stability analysis, where the shear surface is assumed to be planar or bowl shaped. The cubic surface, which is an elliptical parabola, returns the lowest RMSE value of all polynomial functions. RMSE values returned in cross-validating low-order polynomials are in the higher end of the RMSE value range.
Natural neighbour and linear triangulation performed moderately well with RMSE values. Poorly sampled areas and areas with sharp changes in elevation tend to be over-smoothed with these algorithms. Nearest neighbour performed poorly, generating a stepped-polygonal surface. Moving average was the worst algorithm returning a very high RMSE value and an unrealistic spatial pattern.

In general, the anisotropic neighbour weighting produced only slightly higher RMSE values, indicating that the Downie Slide data set does not feature a distinct anisotropic trend in the shear surface geometry. Anisotropy should, in concept, be easily applied to simplistic geometries. For instance; a planar slope would have better anisotropic results if strike parallel data trends were weighted higher than dip parallel data trends. Given a more complex geometry directional trends may simply not exist, or they may be more difficult to apply as anisotropy could exist on multiple scales. For instance; an undulating shear surface may have anisotropy related to trends in regional folding, and then at the landslide boundary the slightly up-warped edges of the shear surface may have some different anisotropic trend related to the nature of the daylighting slip surface.

2.10 Discussion

A series of spatial prediction algorithms have been tested for interpreting the three-dimensional geometry of the basal slip surface at Downie Slide. A number of approaches have demonstrated that geological surfaces can be reasonably interpreted using spatial statistics and interpolation algorithms, while several approaches have proven to be of limited use for this landslide configuration.
2.10.1 Well suited

A number of spatial predictors do perform reasonably well for interpreting shear surface geometry, including continuous surfaces generated by minimum curvature, kriging and some radial basis functions, and a discontinuous interpretation using minimum curvature. Care must be taken to test user defined input parameters to minimize RMSE and avoid excessive undulations and peaks in the resulting surface geometries.

2.10.2 Poorly suited

A number of the poorly performing spatial predictors include inverse distance weighting, nearest neighbour, natural neighbour, moving average and linear triangulation. The poor performance of these methods is likely contributed to by their inability to interpolate values outside the minimum and maximum values of the sample data. Therefore to use these interpolators, the sample set should in theory include the highest and lowest points on a shear surface. As this expectation is not reasonable, these spatial prediction techniques are not suitable for interpreting shear surface geometries. These techniques would perform better with dense, regularly spaced sample data, which are generally not available for a landslide.

In this study, low-order polynomials have been deemed unsuitable due to the complexity of the slip surface shape, however in other applications these algorithms may perform better. Low-order polynomials are applicable when the geological setting justifies a very simple geometric result such as planar, smooth, or spoon-shaped.
2.11 Conclusion

Based on statistical comparison and visual assessment results, the best-suited interpolators for the Downie Slide data set have been narrowed to continuous (smooth) surfaces generated using the minimum curvature (internal tension = 0, boundary tension = 1) algorithm, universal kriging, and the multiquadratic function of the radial basis group and a discontinuous (stepped) surface generated using the minimum curvature (internal tension = 0.5, boundary tension = 1) algorithm. Small-scale variations are evident between the continuous surfaces generated using the minimum curvature, kriging, multiquadratic algorithms, which all visually appear realistic. The discontinuous minimum curvature surface creates the distinct scarp feature with a flatter lower shear along the south side of the slope. All of these geometries improve on the elliptical parabola interpretation which represents the simplified approach traditionally used in three-dimensional slope stability analysis.

Geotechnical analysis of the behaviour of massive landslides is limited when shear surface geometry is simplified by analysis of two-dimensional sections through a slide mass or a three-dimensional geometry with spherical or bowl-shaped slip surfaces. The presented methodology for interpreting the more complex geometry of sub-surface shear interfaces in three-dimensions improves the geomechanical analysis of massive landslide systems. Multiple, and different, surface geometries can be interpreted for any given data set. Small- and large-scale geometric discrepancies result from the use of different spatial predictor techniques and geological assumptions of surface continuity, respectively. No single interpretation technique is suitable for all applications, and the most appropriate algorithm must be selected based on statistical comparison and assessment of spatial
pattern reliability. Ongoing research utilizes three-dimensional numerical models to simulate, observe and compare landslide behaviour. Geometries which closely reproduce measured slope deformations may then be taken as the truest three-dimensional interpretation of sub-surface shear interfaces.

2.12 Acknowledgements

This work has been conducted under the financial support of the Natural Sciences and Engineering Research Council of Canada (NSERC), GEOmatics for Informed DEcisions (GEOIDE), and the Provincial Research Excellence Award Program (PREA). The authors would like to thank BC Hydro personnel, in particular John Psutka and Martin Lawrence for provision of the case history data.

2.13 References


Department of Energy, Mines and Resources (1977) Her Majesty the Queen in right of Canada.


CHAPTER 3

Three-Dimensional Numerical Simulations of the Downie Slide to Test the Influence of Shear Surface Geometry and Heterogeneous Shear Zone Stiffness

3.1 Abstract

Massive, slow moving landslides often exhibit deformation patterns which vary spatially across the landslide mass and temporally with changing boundary conditions. Understanding the parameters controlling this behaviour; such as heterogeneous material properties, complex landslide geometry and the distribution of ground water, is fundamental when making informed design and hazard management decisions. This paper demonstrates that significant improvements to the geomechanical analysis of massive landslides can be achieved through rigorous, three-dimensional numerical modelling. Simulations of the Downie Slide incorporate complex shear zone geometries, ...

* This Chapter has been submitted for publication as:
Kalenchuk K.S., Diederichs, M.S. and Hutchinson, D.J., 2010. Three-dimensional numerical simulations of the Downie Slide to test the influence of shear surface geometry and heterogeneous shear zone stiffness. Submitted to Computational Geosciences manuscript # COM368. 24 manuscript pages.
multiple water tables and spatial variation of shear zone stiffness parameters to adequately reproduce real slope behaviour observed through an ongoing site monitoring program. These three-dimensional models are not hindered by shortfalls typically associated with two-dimensional analysis, for example the ability to accommodate lateral migration of material, and they out-perform more simplified three-dimensional models where bowl-shaped shear geometries are incapable of reasonably reproducing observed deformation patterns.

3.2 Introduction

The study of massive, slow moving slope instabilities has become increasingly important as modern infrastructure and societal developments in mountainous, landslide prone terrain become more common in response to expanding population, and demand for engineered structures such as pipelines and hydro-electric facilities. A complete landslide study should include a detailed assessment of geology, geomorphology, topography, ground water conditions and slope deformation. This generally requires rigorous site investigation including, but not limited to; geological and geomorphological mapping, aerial photography and digital elevation modelling using precise survey techniques such as LiDAR (Light Detection and Ranging), and sub-surface investigation by, for example, borehole drilling, adit excavation and geophysical surveys. Further, slope monitoring programs are crucial for recording spatial and temporal variability in landslide deformation patterns and ground water conditions.

When movements are slow and a landslide is massive, the complete stoppage of movement is difficult to achieve, if not impossible. The most reasonable approach to landslide hazard management may be to live with instability, understanding the associated
risks. Hazard mitigation can then be accomplished through continuous monitoring and analysis of slope behaviour in order to assess trigger scenarios that may initiate slope accelerations. An understanding of slope mechanics can be applied to the risk cost-benefit assessment of engineered mitigation techniques, such as the development of slope drainage infrastructure.

Ultimately, the key to managing hazards posed by massive slow moving landslides is improving the understanding of slide behaviour through studies of slide geomechanics and slope kinematics. Slope behaviour is influenced by a number of factors including material strength, landslide geometry and the distribution of ground water. Spatial heterogeneity and temporal variations in these factors cause massive landslides to exhibit complicated displacement patterns where the magnitude and orientation of observed deformation vary, through space and time, across the landslide mass. Knowledge of factors controlling these spatial and temporal changes in slide kinematics is necessary to make informed predictions of future behavioural responses to various trigger scenarios.

This paper reports a rigorous numerical modelling study; looking at how the spatial complexities in behaviour are primarily controlled by the three-dimensional slip surface geometry and heterogeneity in geomechanical shear zone properties. Simplified cross sections and bowl-shape failure surfaces with homogeneous material properties are often utilized in slope stability analysis. A simplified two-dimensional model is incapable of accounting for lateral migration of material; this assumption is fine for simple translational slides but is otherwise incorrect. Bowl-shaped three-dimensional surfaces can allow lateral migration, but otherwise lack the geometric complexity necessary for modelling spatial variation in slide behaviour, often limiting the entire slope to move as
one mass with uniform deformation. While these limitations have been addressed through many recent numerical modelling advances (for example: Stead et al. 2006, Eberhardt et al. 2002, Stead and Coggan 2006), more emphasis must be given to the importance of three-dimensional landslide simulations capable of accounting for mechanical factors including landslide geometry, heterogeneous material properties, and appropriate ground water conditions. Three-dimensional analyses require increased time and effort to reasonably define these mechanical factors. This often involves added cost in site investigation and a more detailed landslide analysis in order to define the full three-dimensional distribution of key modelling parameters. However, in large, complex slopes, this rigor is capable of significantly improving the understanding of slope mechanics and adds substantial value to making informed decisions for design and hazard management.

Using the Downie Slide case history, this paper demonstrates the application of mixed continuum-discontinuum three-dimensional numerical models to explore the sensitivity of slope behaviour to variable shear surface geometries and the influence of spatial variation in shear zone stiffness parameters as a function of shear zone thickness. This research is complemented by work that gives consideration to the role of internal shear zones and the interaction between primary and secondary failure zones in massive landslides (Kalenchuk et al. 2010a - Chapter 4) and further research into how changing ground water conditions trigger temporal variations in the behaviour of slow moving slopes (Kalenchuk et al. 2010b – Chapter 6). These studies compare simulated slope deformation to landslide behavior observed through slope monitoring data. Trained
models capable of reproducing field observations can be used to improve landslide hazard management by testing trigger scenarios and mitigation techniques.

3.3  **Downie Slide Case Study**

Downie Slide is located 64 km north of the Revelstoke Dam on the west bank of the Revelstoke Reservoir, in the Columbia River Valley, British Columbia, Canada (Figure 3.1). According to the Cruden and Varnes (1996) landslide classification scheme, it is a massive, active, composite, extremely slowly moving rockslide. With an estimated volume of 1.5 billion cubic meters, this rockslide extends 2400 m along the river valley, measures 3300 m from toe to head scarp, and reaches a maximum thickness of about 245 m (Enegren and Imrie 1996). Slope instability is believed to have initiated 9,000 to 10,000 years ago during the last glacial retreat (Piteau et al. 1978, Brown and Psutka 1980). The modern Downie Slide is a composite rockslide where a number of landslide zones have been identified based on morphological features and spatially discriminated slope behaviour and failure mechanisms (Kalenchuk et al. 2010a - Chapter 4).

Downie Slide is found within a metasedimentary sequence which overlies the Monashee Core Complex (Read and Brown 1981, Scammel and Brown 1990 and references therein, Armstrong et al. 1991, Johnston et al. 2000) and is truncated to the east by the Columbia River Fault Zone (Figure 3.1). The highly fractured landslide mass is composed of inter-layered schist, gneiss and quartzite (Imrie et al. 1991), a well developed mica foliation dips down-slope towards the east, and landslide shear zones predominantly follow weak mica rich horizons. In-depth studies of the geological setting have been done by Brown and Psutka (1980), Jory (1974), Wheeler (1965), and others; readers are referred to these
works for a detailed explanation of the local and regional geological and structural setting.

Figure 3.1: (top left) Location map of Downie Slide located in southeastern British Columbia, Canada (modified after Kalenchuk et al. 2009a – Chapter 2), (top right) schematic of Downie Slide geological setting and morphological regions (Kalenchuk et al. 2010a - Chapter 4 after Piteau et al. 1978, Brown and Psutka 1980, Patton and Hodge 1975), (bottom) morphological zoning defined by distinct morphological features and specific slope behaviour (Kalenchuk et al. 2010a – Chapter 4).
3.3.1 Shear Zone Geometry

Geotechnical analyses of massive landslides generally incorporate simplified models of either two-dimensional sections through a slide mass, or three-dimensional geometries with spherical or bowl-shaped slip surfaces. Previous studies of Downie Slide have utilized representative two-dimensional cross sections of a monolithic slide mass (for example Enegren 1995, Kjelland 2004). The irregular nature of displacements in large, slow moving landslides is an indication that the geometry of landslide shear surfaces is more complex and variable in three-dimensions than such simplified models represent (Hutchinson et al. 2006, Agliardi et al. 2001). To improve landslide analysis, a methodology has been developed by Kalenchuk et al. (2009a – Chapter 2) for interpreting the three-dimensional geometry of landslide shear surfaces.

The true geometries of shear surfaces within a landslide are largely unknown and must be interpolated from data obtained from surface and sub-surface site investigation programs. Interpolation is achieved using spatial prediction techniques. A well suited interpolation algorithm must be determined for a specific landslide data set based on geostatistical goodness-of-fit, defined through cross-validation, and expert judgment stemming from an understanding of the local geological setting. Multiple, different, surface geometries may be interpreted as equally valid for any data set (Kalenchuk et al. 2009a – Chapter 2). Geometric analysis by Kalenchuk et al. (2009a – Chapter 2) resolved four geologically reasonable interpretations of the basal slip surface, or principal shear zone, at Downie Slide. These interpretations assume (1) a continuous shear surface between the landslide boundary and borehole intercepts, (2) a stepped surface where the scarp observed along the west and south slide boundaries may extend to depth below the topographic surface
and, for comparison purposes, (3) a simplified interpretation defined by a best fit elliptical parabola. These interpreted shear surfaces demonstrate large-scale geometric discrepancies between the continuous (geometries a, b and c), stepped (geometry d), and simplified (geometry e), while small-scale geometric variations distinguish each of the continuous surfaces (Figure 3.2).

Figure 3.2: (top) Three-dimensional geometries (looking northwest) of the Downie Slide basal slip surface for (1) continuous (a. continuous minimum curvature (smooth), b. kriging of a variogram model and c. the multiquadratic radial basis function), (2) stepped (d. minimum curvature (discontinuous)), and (3) simplified (e. the elliptical parabola) interpretations (modified from Kalenchuk et al. 2009a – Chapter 2). (bottom) Exploded view of a three-dimensional Downie Slide model.
3.3.2 Shear Zone Character

The basal slip surface at Downie Slide is a single gouge-bearing zone ranging from less than 2 to nearly 50 m in thickness. It is located up to 245 m below and sub-parallel to the topographic surface (BC Hydro 1974). Secondary shears do occur within the slide mass, however, based on inclinometer data it is concluded that most displacements occur on the basal slip surface. Shears at Downie Slide are characterized as zones of foliation parallel joints, closely to very closely spaced, with areas of fractured and crushed material, clay and mica gouge and chlorite altered clay (Bourne et al. 1978, Bourne and Imrie 1981, Gerraghty and Lewis 1983).

3.3.3 Slide Behaviour

Slope monitoring at Downie Slide has been ongoing for more than 35 years using a network of instrumentation including survey monuments, inclinometers, piezometers and extensometers. The landslide is active and, according to the Cruden and Varnes (1996) classification scheme, extremely slow moving with a few localized zones which are very slow moving. Slope behaviour is spatially discriminated; the direction and magnitude of observed displacements vary from point to point across the landslide. This paper explores, through numerical modelling, the hypothesis that these inconsistencies are primarily controlled by the three-dimensional slip surface geometry and spatial variation in geomechanical shear zone properties. A detailed discussion of landslide zoning based on observed behaviour and slope morphology is provided by Kalenchuk et al. (2010a – Chapter 4). Temporal variations also occur in response to changing ground water conditions at Downie Slide; the influence of piezometric levels on slope behaviour is being addressed through ongoing research.
In this study, deformation measured between 1990 and 2003 has been taken to represent steady state behaviour as no significant water table fluctuations occurred during this period. Assuming continuous movement across the entire landslide, Figure 3.3 illustrates contoured annual deformation rates observed on the topographic surface by total station survey monitoring data, these have been standardized to values of standard deviation about the global mean. Discrepancies in the direction of deformation are illustrated in Figure 3.4; vectors have been scaled according to annual displacement rates.

Figure 3.3: Contoured deformation rate standard deviations about the global mean; higher rates of movement occur near the head scarp and the central toe of the slide, while the middle portion of the slope is characterized by lower velocities (modified after Kalenchuk et al. 2010a – Chapter 4).
3.4 Numerical Modelling

The numerical modelling carried out in this study incorporates mixed continuum-discontinuum methods (Cundall and Hart 1993, Jing and Hudson 2002) using 3DEC (3-Dimensional Distinct Element Code) (Itasca Consulting Group, Inc. Minneapolis, Minnesota 2003). Model behaviour is calculated by the Lagrangian finite difference method, using an explicit time domain algorithm to solve the equations of motion and incorporate constitutive equations at each time step. The landslide and the undisturbed in-situ material are represented by deformable continuum blocks which are discretized into constant strain-rate elements of tetrahedral shape (Itasca Consulting Group, 2003), while discrete discontinuities define the shear zone. This mixed continuum-discontinuum approach is necessary in order to handle large strains along shear zone interfaces and to avoid unrealistically high stress points at non-smooth or irregularly shaped landslide
boundaries where rigid materials may tend to hang-up rather than incur small
deformation. This section provides a brief overview of landslide model development;
more detail on modelling theory and parametric sensitivity testing is provided in
Appendix B.

3.4.1 Model Geometry

Each numerical landslide model is geometrically generated as a series of columns defined
laterally by user specified grid resolution, and vertically by the topographic elevation
taken from a digital elevation model and a model base datum (Figure 3.5). In this
application, the grid resolution has been set at 100 m, based on grid resolution sensitivity
testing (Appendix B). Each of the columns is individually cut by numerical joints
elements defining the local elevation and orientation of shear surfaces (Figure 3.6). To do
this, each column is first divided into four triangular sub-columns and an elevation
coordinate is interpolated to define where each vertical edge of a sub-column is
intersected by the shear surface. A joint elements is then defined as the plane connecting
the three edge intercepts on each sub-column. During shear surface generation, the
material above and below each joint element is assigned region numbers such that the
landslide material and the undisturbed mass below the landslide can be easily
distinguished within the numerical model. Blocks are joined according to region number
and discretized to simulate a continuum material. Figure 3.7 illustrates the Downie Slide
model.
Figure 3.5: Development of landslide model topography. From left to right: one column, height defined by topographic elevation and column width defined by user specified grid resolution, several columns and a completed topographic model.

Figure 3.6: To define the local elevation and orientation of shear surfaces, each column is subdivided into sub-columns which are independently cut by joints.

Figure 3.7: Downie Slide model (a) looking northwest, (b) typical W-E cross section (Kalenchuk et al. 2009b).
3.4.2 Meshing

The entire landslide model is discretized using tetrahedral-shaped finite difference zones in order to achieve a deformable continuum. For the rockmass matrix located below the landslide, the average edge length of the tetrahedral zones is equal to the grid size. This low mesh density is deemed adequate because the material below the slide does not deform or experience displacements, and a lower mesh density avoids issues with runtimes and memory availability. The blocks within the landslide are discretized with average tetrahedral edge length equal to one quarter the grid size (25 m) which has been determined through mesh size sensitivity testing (Appendix B).

3.4.3 Material Properties and Constitutive Models

Continuum materials in the three-dimensional models are considered to behave elastic-perfectly-plastic. The 3DEC Mohr-Coulomb constitutive model is applied, where the failure envelope consists of the Mohr-Coulomb criterion (Figure 3.8) controlled by cohesion ($c$) and friction ($\phi$)

$$\sigma_1 = \sigma_3 N_\phi + 2c\sqrt{N_\phi}$$

(3.1)

with tension cut off

$$\sigma_3 = \sigma_t$$

(3.2)

Where

$$N_\phi = \frac{1 - \sin(\phi)}{1 + \sin(\phi)}$$

(3.3)
The non-associated flow rule is applied when shear failure occurs and the stress point is set on the curve $f^s = \theta$ (Figure 3.8) using a potential function ($g_s$)

$$g_s = \sigma_1 - \sigma_3 N_{\psi} \quad (3.4)$$

Where

$$N_{\psi} = \frac{1 - \sin(\Psi)}{1 + \sin(\Psi)} \quad (3.5)$$

and $\psi$ is the dilation angle.

When tensile failure occurs, no flow rule is applied and the stress state is set according to $f^t = 0$.

![Figure 3.8: Mohr-Coulomb failure criterion in 3DEC (after Itasca Consulting Group, Inc. 2003).](image)

Downie Slide is primarily composed of inter-bedded schist and gneiss and given that failure through intact material would most likely occur along foliation planes, material properties have been estimated that reasonably characterize schistose material. The intact material below the landslide mass has been assigned a rockmass modulus of 10 GPa,
Poisson’s ratio of 0.25, friction angle equal to 46° and 2700 kg/m³ density. The cohesion and tensile strength of the in-situ material below the slide has been assigned unrealistically high values of 1 GPa and 100 MPa, respectively, to avoid yielding and high plastic strains through in-situ material. The scope of this modelling is to understand behaviour of the material within the landslide rather than the propagation and development of new landslide features beyond the current extent of the modern Downie Slide boundaries.

The disturbed landslide material is characterized to be significantly weaker and less stiff than the intact, in-situ material below the landslide. A long history of slope deformation has resulted in significant losses in rockmass integrity. The landslide mass is characterized by an equivalent rockmass modulus of 500 MPa, Poisson's ratio of 0.25, friction angle equal to 34°, cohesion equal to 1 MPa and tensile strength of 40 kPa. The strength parameters assigned to the numerical models are summarized in Figure 3.9.

The dilation parameter for both in-situ and disturbed materials is set to zero in order to avoid artificial confinement. This assumption is reasonable for the landslide material because the rockmass is already disturbed and assumed to be at residual strength. Furthermore if any failure were to occur through fresh rock within the landslide it would be primarily occurring along weak mica-rich horizons parallel to foliation and given the low quality of these units, negligible dilation would be anticipated. Zero dilation assigned to the in-situ material is considered reasonable as this parameter has no influence on the overall model behaviour since failure does not occur through the intact rockmass.
The in-situ and landslide materials interact along discrete discontinuities generated in 3DEC as joints according to pre-defined shear surface geometries. A perfectly plastic Coulomb-slip constitutive model applied to joint elements considers shear and tensile failure, and dilation (Itasca Consulting Group, Inc. 2003). Elastic behaviour is governed by normal stiffness ($K_n$), shear stiffness ($K_S$), displacements and acceleration, and joint tensile and cohesive strengths are maintained following failure. The Downie Slide shear zone is well developed and the shear is assumed to be perfectly plastic because slow, continuous movements are observed, rather than more brittle behaviour that would otherwise be represented by loss of cohesive and tensile strength at yield.

The maximum allowable tensile force acting on an intact joint controlled by joint tensile strength (5 kPa) and sub-contact area ($A_c$) is defined as:

$$T_{\text{max}} = -TA_c$$  \hspace{1cm} (3.6)

Cohesion ($c$) and friction ($\varphi$) govern the maximum shear force acting on an intact joint as per:

$$F_{\text{max}}^S = cA_c + F'' \tan \varphi$$  \hspace{1cm} (3.7)
Contact forces are corrected at the onset of failure. When tensile strength is exceeded, the normal and shear forces are set to zero and when failure in shear occurs, the shear force is set to

\[ F_i^s := F_i^s \frac{F_i^{s\max}}{F_i^s} \quad (3.8) \]

where \( F_i^s \) is the shear force magnitude equivalent to the absolute value of the shear force vector.

Through a series of sensitivity tests (Appendix B) friction and cohesion values for the shear surfaces have been approximated to be 19º and 400 kPa, respectively. In theory, it is unrealistic to apply a single set of material parameters to the full spatial extent of a massive landslide. Rockmass and shear zone heterogeneity should be taken into account such that models are able to consider spatial variation in factor of safety. However, in the absence of an extensive laboratory testing program, no data is available to justify the application of any spatial distribution in friction and cohesion to the numerical simulation of Downie Slide shear surfaces.

The joint dilation angle is assumed to be zero. This assumption is related to the fact that the Downie Slide is thought to have been moving for a long period of time and as such the shears are well developed. Dilation occurs when shearing takes place along a fresh joint. As shearing continues, asperities are worn down and eventually, when residual friction is achieved, dilation no longer occurs with progressive displacements. 3DEC takes dilation into account as a function of shear displacement and the dilation angle (\( \psi \));

\[ \Delta U^s(\text{dil}) = U^s \tan \psi \quad (3.9) \]
If dilation were non-zero, normal forces would be corrected during joint displacement using:

\[
F^n := F^n + K_n A_c \Delta U^S \tan \psi \quad (3.10)
\]

In 3DEC, shear zones are represented by discrete discontinuities with zero thickness, however, shear zones, by nature, have some magnitude of thickness. If left unaccounted for, this thickness inconsistency would influence the mechanics of landslide modelling. Intuitively, a shear zone composed of multiple fractures, gouge and brecciated material will be less stiff than a discrete joint. Therefore varying contact stiffness parameters are used to account for the thickness discrepancy.

3DEC recognizes discrete contacts using a cell mapping routine and a user specified tolerance parameter. When new contacts are recognized, the physical contact between adjacent blocks is numerically replicated as a data element describing contact information such as strength parameters, the common plane unit normal and active forces. 3DEC contacts are soft, meaning that the measurable contact stiffness is represented as finite normal stiffness. Hard contacts, in contrast, use algorithms to avoid block penetration.

Block faces at a contact are discretized into sub-contacts, which are created for each surface node on a deformable block. Sub-contacts are used to track contact conditions such as forces, sliding and block separation. Sub-contacts on opposing faces calculate relative displacements and forces using contact logic described by a set of parallel springs. The relative velocity \( (V_i) \) across a sub-contact is obtained from the sub-contact velocity and the velocity of the opposite face. Incremental displacements \( (\Delta U_i) \) over time \( (t) \) are then:
\[ \Delta U_i = V_i \Delta t \] (3.11)

which can be resolved into normal and shear components according to the sub-contact unit normal. All sub-contacts on a common face are assigned a common unit normal taken as the unit normal to the common plane. Incremental displacements are used to calculate the normal and shear elastic force increments:

\[ \Delta F^n = -K_n \Delta U^n A_c \] (3.12)
\[ \Delta F^s = -K_s \Delta U^s A_c \] (3.13)

where \( K_n \) and \( K_s \) are normal and shear stiffness, respectively, and \( A_c \) is the sub-contact area. Elastic force increments are adjusted according to contact constitutive relations and then are used to update the contact total force vector. In deformable blocks, forces are resolved at vertices and added to other gridpoint forces.

A number of methods have been suggested for estimating contact normal and shear stiffness values in jointed models, for instance; as a function of bulk modulus, shear modulus and the size of zones adjoining the joint in the normal direction (Hart 1993), roughly estimated as a function of typical system stress and joint normal displacement, where joint normal displacement should not exceed the typical zone size by more than 10% (Duncan and Goodman 1968), or by empirical formulations (for example Bandis et al. 1983). Typical values for joint stiffness range from < 10 GPa/m for weathered rock or soft clay infillings, to 100 GPa/m for tight clean joints in strong rock (Hart 1993, Bandis et al. 1983). Given the nature of this application where joints are simulating shear zones rather than discrete fractures, the joint shear and normal stiffness have been assigned base case values of 50 and 100 MPa/m, respectively. Spatial variation in these base case...
values has been simulated to explore how the mechanical slope behaviour is influenced by the stiffness as a function of the Downie Slide basal slip surface shear zone thickness.

### 3.4.4 Boundary Conditions

The bottom and vertical edges of the model are constrained by velocities equal to zero in the direction normal to the model face. This boundary condition acts as a roller-boundary, allowing displacements parallel to, but not normal to, the model edges.

### 3.4.5 Pore Pressure

As illustrated in Figure 3.10, multiple shears and multiple water tables have been interpreted at Downie Slide (Kalenchuk et al. 2009c). In this modelling application, only the basal slip surface has been simulated; as such the lower water table is applied to material below the landslide and the shear surface interface, and the upper water table is applied to material within the landslide. FISH (a programming language embedded within 3DEC) routines have been developed to apply specific water tables to all gridpoints (for continuum materials) and sub-contacts (for discontinuum elements). Pore-pressures are calculated by linearly interpolating the depth of water at each gridpoint location based on the local elevation of the appropriate piezometric surface. It should be noted that a built-in *water table* command is available in 3DEC; however it is not capable of handling multiple water tables in a single model, as such, coding FISH routines significantly improves water table modelling versatility. It should also be noted that a model must be cycled at least one time step prior to assigning sub-contact pore pressures. This is because 3DEC does not discretize contacts into sub-contacts prior to the first time step, and pore pressures must be assigned directly to sub-contacts. Further, when deformation
occurs along the shear zone, and joint slip occurs within the landslide model, sub-contacts are continually created and destroyed. The default pore-pressure is zero for newly created sub-contacts, and it is therefore necessary to routinely update sub-contact pore pressures.

![Diagram of water tables and landslide](image)

Figure 3.10: Multiple water tables have been identified at Downie Slide (Kalenchuk et al. 2009c). For numerical simulation, the lower water table (confined below the basal slip surface) is applied to the basal slip surface and material below the basal slip surface, the upper water table is applied to material within the landslide.

### 3.4.6 Buttress Load of a Full Reservoir

For water table conditions associated with a full reservoir, a toe load is applied to account for the buttressing effect on the inundated portion of the slope. The load, as depicted in Figure 3.11, is normal to the topographic surface and equivalent to the hydrostatic load of the reservoir water \( F \), which is a function of water density \( \rho \), gravity \( g \), the depth of water \( h \), and the area of the toe \( A \).

\[
F = \rho ghA
\]  

(3.14)

This force, applied normal to the topographic surface, is resolved to north, east and downward components based on the local topographic slope of about 14° towards 065°NE.
\[
\begin{bmatrix}
F_{\text{north}} \\
F_{\text{east}} \\
F_{\text{down}}
\end{bmatrix}
= 
F 
\begin{bmatrix}
\sin(14)\sin(65) \\
\sin(14)\cos(65) \\
\cos(14)
\end{bmatrix}
\] (3.15)

Figure 3.11: A toe load is applied to account for the buttressing effect of the reservoir.

### 3.4.7 In Situ Stresses

The vertical to horizontal stress ratio is assumed to be equal to one. In situ stresses are initialized by assigning zone stresses as a function of rock density, and depth below the topographic surface and gravity.

### 3.4.8 Damping

Local damping is applied to the continuum within deformable blocks to absorb kinetic energy. The force on a node is damped proportional to the magnitude of the local unbalanced forces, and the amount of damping varies from point to point. The direction of the damping forces is such that energy is always dissipated and body forces vanish for steady state conditions.
3.4.9 Numerical Procedure

A number of set-up stages have been developed in order to avoid shocking the numerical models while achieving initial equilibrium. Table 3.1 summarizes the consecutive stages run to equilibrium, where equilibrium is defined as the ratio of maximum unbalanced force in the model to the maximum zone force (where zone force is the average zone stress multiplied by area) equal to less than 1%. Between each stage, grid point displacements and velocities are initialized to zero. Once all of the set-up stages have achieved equilibrium all grid point displacements and velocities are again initialized to zero and models are run for a set number of time steps. Different simulations are compared on the basis of total displacements achieved over this defined time-stepping period.
Table 3.1: Summary of staged set-up for minimizing initial deformation.

<table>
<thead>
<tr>
<th>Stage</th>
<th>Material</th>
<th>Constitutive Model</th>
<th>Rockmass Modulus (GPa)</th>
<th>Poisson's Ratio</th>
<th>Density (kg/m³)</th>
<th>Cohesion (MPa)</th>
<th>Tensile Strength (MPa)</th>
<th>Friction (°)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>In situ and landslide</td>
<td>Linear-elastic</td>
<td>1000</td>
<td>0.49</td>
<td>2700</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>No shear zone is implemented. Grid point pore pressures and in-situ stress are initialized. High elastic properties prevent large deformation. High Poisson's ratio allows lateral deformation of elements during the redistribution of initial stress concentrations.</td>
</tr>
<tr>
<td>B1</td>
<td>In situ and landslide</td>
<td>Linear-elastic</td>
<td>100</td>
<td>0.35</td>
<td>2700</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>The elastic properties of the rockmass are reduced, there is no shear implemented.</td>
</tr>
<tr>
<td>B2</td>
<td>In situ and landslide</td>
<td>Linear-elastic</td>
<td>10</td>
<td>0.25</td>
<td>2700</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Elastic properties of the rockmass are reduced to realistic value. There is no shear implemented.</td>
</tr>
<tr>
<td>C</td>
<td>In situ and landslide</td>
<td>Elastic-plastic, Mohr-Coulomb</td>
<td>10</td>
<td>0.25</td>
<td>2700</td>
<td>1000</td>
<td>100</td>
<td>46</td>
<td>The shear zones are implemented. High rock cohesion is applied to avoid plastic deformation and significant changes in the position of gridpoints.</td>
</tr>
<tr>
<td>C2</td>
<td>In situ and landslide</td>
<td>Elastic-plastic, Mohr-Coulomb</td>
<td>10</td>
<td>0.25</td>
<td>2700</td>
<td>1000</td>
<td>100</td>
<td>46</td>
<td>Sub-contact pore pressure is applied to shear surfaces.</td>
</tr>
<tr>
<td>D1</td>
<td>In situ</td>
<td>Elastic-plastic, Mohr-Coulomb</td>
<td>10</td>
<td>0.25</td>
<td>2700</td>
<td>1000</td>
<td>100</td>
<td>46</td>
<td>Material properties for the landslide mass are adjusted to approach more realistic values.</td>
</tr>
<tr>
<td>Landslide</td>
<td>Elastic, plastic, Mohr-Coulomb</td>
<td>1</td>
<td>0.25</td>
<td>2700</td>
<td>1</td>
<td>0.05</td>
<td>38</td>
<td></td>
<td></td>
</tr>
<tr>
<td>D2</td>
<td>In situ</td>
<td>Elastic, plastic, Mohr-Coulomb</td>
<td>10</td>
<td>0.25</td>
<td>2700</td>
<td>1000</td>
<td>100</td>
<td>46</td>
<td>The material properties of the landslide mass are reduced to realistic values.</td>
</tr>
<tr>
<td>Landslide</td>
<td>Elastic, plastic, Mohr-Coulomb</td>
<td>0.5</td>
<td>0.25</td>
<td>2700</td>
<td>1</td>
<td>0.04</td>
<td>34</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

112
3.4.10 Comparison of Field and Simulated Data

Deformation measured on the topographic surface of the numerical models is compared to field survey monuments on the basis of relative deformation rates and direction of displacements. Deformation data is sampled in numerical models at the same geographic location as field instruments to ensure consistent data location and density. Field data describes displacements per time while model data provides displacements per number of time steps. It is therefore necessary to standardize slope deformation by plotting contoured displacement rates on contour intervals which correspond to the average and standard deviation of a given data set (Figure 3.3). Quantitative comparison is achieved using $R^2$ values, and qualitative assessment of spatial patterns projected on contoured rate plots is done visually (Figure 3.12).

The direction of displacements in numerical models are compared to field data according to cosine values defined by the north and east components of total displacement. The cosine value ($c_i$) at data point $i$ is defined as:

$$c_i = \frac{(e_{fi} \times e_{mi} + n_{fi} \times n_{mi})}{\sqrt{(e_{fi}^2 + n_{fi}^2)} \times \sqrt{(e_{mi}^2 + n_{mi}^2)}}$$  \hspace{1cm} (3.16)

where $(e_{fi}, n_{fi})$ and $(e_{mi}, n_{mi})$ are the north and east components of movement measured at point $i$ for the field and model data sets, respectively. A cosine value of 1 indicates that the modelled deformation at that point is in the same direction as the observed field deformation, lower cosine values indicate poorer correlation between simulations and field observations, with a value of 0 indicating perpendicular direction of deformation.
Figure 3.12: (top) Displacement rate standard deviation from the mean measured in the field and modelled returns $R^2$ values used to quantitatively compare various model simulations (bottom) Contoured displacement rate standard deviations for visual comparison between modelled and measured data.
3.5 Results and Discussion

3.5.1 Shear Geometry

To assess the influence of large- and small-scale geometric variations on simulated slope behaviour, a series of numerical models have been developed for the predicted geometries (Figure 3.2). These simulations explore how much detail in shear surface geometry is required to adequately reproduce observed slope behaviour and investigate if rigorous geometric interpretations are necessary in the geomechanical analysis of large landslides.

Figure 3.13 illustrates the contoured displacement rates for each of the five shear surface geometries defined by Kalenchuk et al. (2009a – Chapter 2). All complex geometries (geometries A to D) represent the field data reasonably well, with the highest movement rates occurring near the head scarp, and the slowest rates found through the central portion of the slide. Statistically, geometries a, b and d perform the best, returning $R^2$ values of 0.73, 0.76 and 0.86 respectively. Localized discrepancies in model results are evident between the continuous and stepped geometries. Most notably, the stepped surface reproduces the appropriate rate gradient between M19 and M43 (Figure 3.3), while the continuous surfaces return displacement rate standard deviations from the mean at M43 which are higher than those observed in the field producing a notable outlier in the $R^2$ data as illustrated in Figure 3.14. This discrepancy is related to the local shear surface geometry; the stepped surface has a shallower slope angle through the upper region than the continuous surfaces (Figure 3.15). The simplified geometry does not produce adequate results, as the spatial pattern of deformation rates varies considerably.
from field observations and the $R^2$ value (0.17) is significantly lower than for the other models.

Figure 3.13: Contoured deformation rates measured in numerical models at the location of field survey monuments. (1) Continuous (a. minimum curvature, b. kriging of a variogram model and c. the multiquadratic radial basis function), (2) stepped (d. minimum curvature), and (3) simplified (e. the elliptical parabola) geometries.

Figure 3.14: Statistical correlation using $R^2$ values to compare displacement rate standard deviation from the mean measured in the field versus modelled geometries.
Figure 3.15: Schematic illustrating that the stepped surface geometry has a shallower grade than the continuous shear surface geometries through the upper portion of the slope (inset shows section location).

In comparing field data (Figure 3.3) to numerical data, it is apparent that in reality the central toe region achieves displacement rates which are, relative to the rest of the slope, higher than those achieved in the modelling. This discrepancy can be accounted for by considering landslide morphology and complex failure processes. The central toe region is an active zone (Patton and Hodge 1975, Piteau et al. 1978, Kalenchuk et al. 2010a – Chapter 4) within the main landslide defined morphologically as a depressional basin bounded by internal scarps. In reality, this zone is influenced by secondary shears which add increased degrees of freedom that are not accounted for in a simulated continuum representation of the landslide mass. Therefore, the rate of movement of the central toe region of numerical models is, relatively, not as fast as the observed field behaviour.

Directions of deformation for each of the simulated geometries are compared to field data in Figure 3.16. All models simulate direction observations reasonably well, returning favorable comparisons for all instruments through the middle and lower portions of the slope. M19 consistently shows poor directional correlation. This discrepancy between the models and field data occurs due to localized deformation mechanisms not accounted
for in the continuum landslide mass. In this upper region of the slope, the real landslide is in a retrogressive regime where large rock blocks are bounded by extensional features. It is probable that these large blocks are subject to rotations and differential displacements.

Figure 3.16: Cosine values depicting correlation in direction of deformation between field and simulated data points for (1) continuous (a. minimum curvature (smooth), b. kriging of a variogram model and c. the multiquadratic radial basis function), (2) stepped (d. minimum curvature (discontinuous)), and (3) simplified (e. the elliptical parabola) geometries.

When numerical simulations are compared to field data, it is apparent that the more complex geometries are necessary to reproduce the deformation observed in reality. Results for simulated shear surface geometries indicate that the small-scale variability has only minor influence on deformation patterns. Large-scale geometric variability plays a more important role than small-scale variability in controlling landslide deformation patterns. While it cannot be concluded that either the continuous or stepped geometry better represents true slope behaviour, it is obvious that the oversimplified bowl-shaped slip surface is inadequate at reproducing observed slope deformation. Results clearly
show that rigorous interpretation of shear surface geometry is necessary for adequate simulation of the Downie Slide. Results also demonstrate that slightly different complex geometries can produce comparably reasonable results, and therefore it may be necessary to complete further slope assessment using multiple geometries, in order to explore landslide mechanics in more detail. Based on the geometric results for Downie Slide, it is concluded that the continuous minimum curvature and krigging algorithms, as well as the stepped minimum curvature function, all return reasonable displacement patterns. These geometries will be used for further assessment of the influence of spatial variation in joint stiffness.

3.5.2 Spatial Variation in Joint Normal and Shear Stiffness

A second series of numerical models looks at the influence of spatially varied joint normal and shear stiffness values. The basal slip surface thickness varies considerably. In this 3DEC application, shears are numerically represented as discrete joint contacts with stiffness parameters defined to simulate a much thicker deformation zone. It is hypothesized that shear thickness correlates to the stiffness of that zone, and so these models test the influence of spatially variable shear zone stiffness on landslide mechanics. This is achieved by dividing the shear zone into three regions representing thin, average, and thick portions of the basal slip surface (Figure 3.17). These regional divisions are required, rather than a continuous scale of shear thickness, because the stiffness parameters are assigned to individual joint materials, for the purposes of modelling, and a continuous range of thickness regions would require the development of an unrealistic number of joint materials.
Figure 3.17: (left) Distribution of shear zone thickness defined from borehole logs. (right) Thickness regions are used to define variation in contact stiffness parameters.

Three sets of runs have been developed with different basal slip surface geometries including the continuous minimum curvature and krigging algorithms, as well as the discontinuous minimum curvature algorithm. Stiffness multipliers are applied to the base case shear and normal stiffness, of 50 MPa/m and 100 MPa/m respectively. For each geometry, the ratio of stiffness parameter multipliers (thin region:average region:thick region) is varied between 0.1:1:10 and 10:1:0.1.

Figures 3.18 through 3.20 demonstrate the influence of spatial variation in shear zone stiffness on displacement rate standard deviations from the mean. Observation of spatial deformation patterns and review of $R^2$ values indicates that a stiffness increase in thin areas and stiffness decrease in thick areas improves the simulation of observed slope behaviour. Similar $R^2$ values, for those models showing the best results, demonstrate that there is very little change in correlation between the displacement rate standard deviations from the mean measured in the field versus modelled with improved distribution of shear zone stiffness parameters. Changes to landslide behaviour are also evident when the stiffness of thin and thick areas are reduced and increased respectively. This scenario, tested for thoroughness, is not mechanically correct and the resulting deformation patterns
correlate poorly with observed field behaviour. In all cases, spatial variation in shear zone stiffness was found to have very little influence on the direction of deformation.

Figure 3.18: Contoured deformation rates for the continuous minimum curvature shear surface geometry at varying ratios of stiffness parameters (a) 0.1:1:10, (b) 0.2:1:5, (c) 0.5:1:2, (d) 1:1:1, (e) 2:1:0.5, (f) 5:1:0.2, (g) 10:1:0.1.
Figure 3.19: Contoured deformation rates for the continuous krigging shear surface geometry at varying ratios of stiffness parameters (a) 0.1:1:10, (b) 0.2:1:5, (c) 0.5:1:2, (d) 1:1:1, (e) 2:1:0.5, (f) 5:1:0.2, (g) 10:1:0.1.
3.6 Summary

The analysis of landslide mechanics is often limited by oversimplified numerical simulations where shear zone geometries are depicted as spoon or bowl-shaped and material strength parameters are assumed to be homogenous. Rigorous model development has achieved fully three-dimensional simulations of the Downie Slide capable of accounting for site specific details such as multiple water tables and the buttressing load of the reservoir, while applying reasonable boundary conditions, in-situ stresses, material properties and constitutive models.
Two series of simulations assess the influence of basal slip surface geometry and the role of spatially variable contact stiffness as a function of shear zone thickness. Numerical models with varying geometries of the Downie Slide basal slip surface have demonstrated that a three-dimensional interpretation of shear zone geometry is necessary to improve simulated slope deformation. The distribution of relative deformation rates and, to a lesser extent, the direction of deformation prove to be sensitive to large-scale geometric variations, while small-scale variability has less influence on simulated slope behaviour. Spatial variation in shear zone stiffness as a function of thickness also influences slope behaviour. The application of heterogeneous stiffness parameters induces localized stress concentrations and joint slip; contributing to complex three-dimensional slope deformation. Small improvements to the simulation of observed slide deformation at Downie Slide are achieved when thicker regions of the basal slip surface are less stiff and thinner regions are stiffer. It can be concluded that the thickness of a shear zone is a controlling factor for slide behaviour; however the absolute scale stiffness variability is not easily defined. Numerical simulations prove to be more sensitive to shear zone geometry than the absolute scale of contact stiffness, however for a rigorous assessment of landslide mechanics both factors must be taken into consideration.

Modelling results presented here have achieved reasonable representation of Downie Slide behaviour observed through slope monitoring data. These three-dimensional models overcome shortfalls of simplified two-dimensional analyses, particularly by allowing for lateral migration of material. Further, the comparison of complex shear zone geometries to a simplified bowl-shaped geometry demonstrates the necessity for detailed model development in order to adequately reproduce observed landslide deformation.
patterns. Building on these results, Kalenchuk et al. (2010a – Chapter 4) use the best geometries (minimum curvature and krigging geometries as well as the stepped minimum curvature geometry) and stiffness distributions (contact stiffness multiplier ratio of 0.2:1:5) to test how landslide morphology zoning and the inclusion of secondary shears influence landslide simulations. Ongoing work explores how temporal changes in slope behaviour are controlled by fluctuating ground water boundary conditions.

3.7 Acknowledgments

This work has been made possible through contributions by NSERC, CFI and GEOIDE. Thanks to BC Hydro, particularly J. Psutka and D. Moore, for site and data access.

3.8 References


Kalenchuk K.S., Hutchinson, D.J. and Diederichs, M.S., 2010a. Morphological and geomechanical analysis of the Downie Slide using 3-dimensional numerical models: testing the influence of internal shears and interaction between landslide regions on simulated slope behaviour. Submitted to Landslides manuscript # LASL 241: 32 manuscript pages.


CHAPTER 4*

Morphological and Geomechanical Analysis of the Downie Slide Using 3-Dimensional Numerical Models: Testing the Influence of Internal Shear Zones and Interaction between Landslide Regions on Simulated Slope Behaviour

4.1 Abstract

Downie Slide has been interpreted as a massive, composite rockslide, and a number of landslide zones have been defined based on the interpretation of morphological features and a detailed assessment of spatially discriminated slope behaviour. Key factors controlling the mechanics of massive slow moving landslides can be interpreted through the observation and detailed study of the slope behaviour and physical characteristics. Once identified, those components influencing slope deformation can be tested using

* This Chapter has been submitted for publication as:
Kalenchuk, K.S., Hutchinson, D.J. and Diederichs, M.S., 2010. Morphological and geomechanical analysis of the Downie Slide using 3-dimensional numerical models: testing the influence of internal shears and interaction between landslide regions on simulated slope behaviour. Submitted to Landslides manuscript # LASL 241. 32 manuscript pages.
three-dimensional numerical models. This study presents two series of numerical simulations which have been developed to test how explicitly defined internal shear zones and the interaction between landslide morphological regions influence the overall landslide behaviour. Results from these numerical simulations, when compared to field monitoring data, indicate that internal shear zones have little influence on Downie Slide deformation while the interaction between morphological zones plays a larger role in slope kinematics.

4.2 Introduction

Assessment of landslide mechanics draws from an understanding of the geological, morphological, hydrogeological and geomechanical setting and observations of slide behaviour through slope monitoring. More than 35 years of slide monitoring data has been analyzed, in parallel with a detailed study of the physical landslide setting, in order to interpret the modern mechanisms of Downie Slide. From this a new hypothesis has been suggested by Kalenchuk et al. (2009a) that Downie Slide, in its modern state, is a massive, active, composite, extremely slow moving rockslide. Since initial proposal of this hypothesis, LiDAR (Light Detection and Ranging) data has been made available to the authors by BC Hydro. This detailed bare-earth topographic data provides new observations of morphological features. Landslide zoning, the division of the landslide mass into regions based on morphological features and spatially discriminated slope behaviour and failure mechanisms, has been revised accordingly.

Kalenchuk et al. (2010 – Chapter 3) provide an overview of a methodology for rigorous three-dimensional numerical modelling of massive, slow moving landslides. Their results demonstrate that variations in the basal shear zone geometry and spatial distribution of
shear zone stiffness parameters influence modelled landslide deformation patterns. Modelling presented in this paper builds model complexity further by considering discrete regions within the landslide mass through the inclusion of secondary, internal shear zones. This is achieved through two series of numerical simulations. The first series of simulations takes the best geometries and shear zone stiffness distributions found by Kalenchuk et al. (2010 – Chapter 3) and incorporates one and two internal shears which have been identified in the borehole geology logs. These shears are assumed to be either (a) continuous across the entire landslide mass or, (b) related to landslide zoning and confined to specific morphological regions. The second series of simulations focus on the interaction between landslide morphological regions and tests how the interactions between primary and secondary failure zones influences simulated slope behaviour.

4.3 Case Study

This study of Downie Slide draws geological, morphological and geomechanical data from a number of sources. Mapping was initiated in 1956 during site investigation prior to reservoir development. The geological and morphological information has since been supplemented by numerous subsequent studies (for example; Wheeler 1965, Jory 1974, BC Hydro 1974, 1976, Bourne et al. 1978, Bourne and Imrie 1981, Brown and Psutka 1980, Gerraghty and Lewis 1983), records of borehole geology logs, geological mapping of two drainage adits, recent site visits by the authors in 2008 and 2009, and LiDAR data acquired in 2009.

Downie Slide, situated on the west bank of the Revelstoke Reservoir in southeastern British Columbia, Canada, is regionally located within a metasediment sequence which overlays the Monashee complex, a high grade metamorphic core complex in the Southern
Omineca belt of the Canadian Cordillera (Read and Brown 1981, Scammel and Brown 1990 and references therein, Armstrong et al. 1991, Johnston et al. 2000). In the vicinity of Downie Slide the metasedimentary sequence is made up of a number of lithological units composed primarily of thinly bedded quartzites, semipelites, psammites, calc-silicates and marbles (Brown and Psutka 1980). The metasediment sequence is truncated by the Columbia River Fault Zone on the east bank of the Columbia River Valley across from the toe of Downie Slide; a foliated biotite granodiorite pluton occupies the hanging wall (Brown and Psutka 1980) (Figure 4.1). Cataclastic fabrics across the Columbia River Fault are evident in a zone up to 1 km thick, more recent brittle faulting has resulted in localized zones of clay and graphitic gouge (Brown and Psutka 1980).

The landslide is located in the Columbia River Fault footwall, within a thick, continuous sequence of pelites, semipelites and minor psammites. The rockmass is highly fractured (Imrie et al. 1991), and subtle variation in the rock type and complex interfingering of the inter-layered schist, gneiss and quartzite make it impossible to correlate lithological units between different boreholes (BC Hydro 1978, Jory 1974). Quaternary sediments include local alluvial and colluvial deposits, till and scattered glaciofluvial sediments (Fulton and Achard 1985).
Three phases of deformation are recognized in the Downie Slide region. The first deformation phase did not significantly influence the structural setting of Downie Slide, while the location and attitude of the second and third phases and their associated fabrics controlled the slide geometry (Brown and Psutka 1980). Phase 1 is displayed as first...
order isoclinal folds with attenuated limbs and primary bedding planes which are parallel to
the axial surface; the fold axis exhibits variable attitudes (Brown and Psutka 1980). Phase 2 deformation dominates the Downie structural setting. Bedding and phase 1 foliations are folded by tight to isoclinal folds with moderately thickened hinges, and attenuated limbs and metamorphic minerals aligned with the fold hinge, as described by Brown and Psutka (1980). Phase 2 deformation has produced a penetrative axial plane mica foliation which dips 20° eastward; the basal shear zone of the landslide has developed parallel to this foliation. Phase 3 deformation is most evident in the area to the west of Downie Slide where third phase major and minor folds, with moderate to steeply westward dipping axial surfaces, have been superimposed on the Phase 2 geometry. These Phase 3 folds die out eastward, and are only of minor significance within the slide mass (Brown and Psutka 1980).

Jointing at Downie Slide is dominated by one gently inclined set, dipping east (J1), and two sub-vertical sets dipping roughly east (J2) and north (J3). J1 is parallel to foliation, the variable attitude averages a 22° dip and 080° dip direction. J1 joints are continuous, closely spaced, smooth, and wavy or planar. They may have micaceous gouge coating and/or surface staining. J2 is smooth and wavy to planar with surface staining, chloritization and clay infilling. J3 joints are planar and smooth to wavy and rough. J2 and J3 are moderately to closely spaced and offset by J1. All three joint sets are well defined in the gneissic rocks. J3 joints do not occur in the mica schist and J1 and J2 joints are poorly defined or widely spaced in the weaker more ductile schist (Gerraghty and Lewis 1983).
Morphological regions of Downie Slide, as shown in Figure 4.1, have been recognized by Piteau et al. (1978) and Patton and Hodge (1975) as; (1) the head area immediately down slope from the head scarp, (2) the central area between the head and the toe of the slide, (3) the south knob area located at the downstream toe, (4) the north knob area located at the upstream toe and (5) the active area located between the south and north knob areas. Patton and Hodge (1975) also note subordinate areas consisting of talus covered slopes along the base of the head and side scarps and a possible shallow slide located half-way down the northern boundary. The morphological regions are largely interpreted from aerial photographs and their boundaries are not precisely delineated. New interpretations of morphological landslide zoning presented in this paper draw from aerial photographs, LiDAR, site visits and slope monitoring data, and give more detailed consideration to failure mechanisms occurring within individual zones and the mechanical interaction between zones.

Slope behaviour is described by data from survey monuments and inclinometers. These track the behaviour of the slope through the period of baseline monitoring, drainage through boreholes drilled from two adits at the base of the slide, reservoir impoundment and subsequent drainage maintenance campaigns. Survey monuments provide a reliable record of surficial deformation rates and direction of movement. Measured surficial movements account for displacement along discontinuities as well as internal strain. Spatial variations in surficial deformation patterns can be influenced by the shape of a failure surface and variable landslide thickness, heterogeneous material properties and differences in state of stress associated with geological history, ground water conditions and previous movement. Inclinometer records provide information on the nature of
deformation through the landslide profile, for instance this data can be used to determine whether displacements reflect slip concentrated along shear zones, or internal deformation throughout the entire landslide mass. Inclinometers have inherent challenges in data interpretation due to sensitivity to instrument drift, spiraled or kinked borehole casings and sensor alignment. While corrections can be applied to known data errors, such as zero-shift errors and rotational errors, data from inclinometers, particularly in deep holes, is best used to provide a sense of where deformation is occurring rather than to develop an exact displacement profile (Mikkelsen 1996). A number of figures throughout the following sections illustrate inclinometer data. This data has been collected, on average, twice per year, and the available data covers the time period from 1971 to 2007. The figures present here illustrate subsets of inclinometer data taken over five equal intervals spanning the operation of each specific instrument. The locations of shear zones, as reported in borehole geological logs and identified as movement zones in inclinometer data, are illustrated; and, where data permits, the thickness of these shear zones is also shown.

Despite early observation of different morphological regions, and instrumentation records demonstrating spatially discretized slope deformation, the Downie Slide has been routinely studied as a monolithic slide mass ignoring the interaction between different morphological regions (Enegren 1995, Kjelland 2004, Kalenchuk et al. 2010 – Chapter 3) and the influence of multiple shear zones. Stability assessments have primarily been completed using two-dimensional landslide cross sections with simplified geometries and singular water tables (Enegren 1995, Kjelland 2004). Recent work by Kalenchuk et al. (2010 – Chapter 3) has taken advantage of improved simulation tools and accelerated
model run times to generate fully three-dimensional numerical simulations capable of accounting for complex shear surface geometries, multiple water tables, and spatial variation in shear zone material properties.

4.4  **Landslide Zoning**

Landslides are defined as composite when different types of movement occur in different areas of the displaced mass (Cruden and Varnes 1996). In order to interpret landslide mechanics, it is necessary to recognize whether different landslide zones exhibit variable behaviour. Figure 4.2 illustrates zones dividing Downie Slide into areas with distinct morphological features and specific slope behaviour. These zones have been largely interpreted from LiDAR data, observations made during recent site visits by the authors in 2008 and 2009, and thorough analysis of slope monitoring data. It is important to recognize that the landslide zoning presented here is based on observations of the modern Downie Slide. It is hypothesized that failure was initiated some 9000-10000 years ago during deglaciation (Piteau et al. 1978, Brown and Psutka 1980), and it must be understood that landslide mechanisms during early stages of the instability may have varied significantly from those observed today.
4.4.1 Landslide Boundary

The most prominent morphological features at Downie Slide are the head scarp and side scarp which mark the southwest and south slide boundaries respectively. These sub-vertical faces reach up to approximately 120 m in height and their geometry, as well as the blocky nature of the scarp faces, is controlled by the distribution of joint sets (two sub-vertical and one sub-horizontal) (Figure 4.3). The current western limit of the head scarp coincides with the hinge zone of a major monoclinal flexural fold associated with Phase 3 regional folding (Brown and Psutka 1980). Since initial landslide recognition, the northern slide boundary has been approximated by vague lineaments mapped on aerial...
photographs and field reconnaissance (Patton and Hodge 1975), and until recently this area of the slope has been poorly understood due to data limitations reflecting limited accessibility to this part of the slide. Newly acquired LiDAR imagery (Figure 4.4) clearly shows the north extent of the landslide mass, confirming that early interpretations of the north boundary have been fairly reasonable. The northeast alignment of this north boundary and the occurrence of streams which flow northeast past the landslide boundary may suggest some structural control on the lineament development.
Figure 4.4: LiDAR image of Downie Slide (left) plan view illustrating side and head scarps and the north landslide boundary (dash line) clearly visible in LiDAR data (right) isotropic views looking northwest (top) and southwest (bottom).

### 4.4.2 The Upper Region

The upper region in Figure 4.2 is characterized by hummocky terrain (Figure 4.5) made up of large partially disturbed rock blocks separated by extensional features which are littered with jumbled talus. Translational retrogressive failure is recognized in this region (Patton and Hodge 1975, Piteau et al.1978, Kalenchuk et al. 2009a). Large blocks are interpreted to have broken off the scarp progressively, thereby increasing the extent of the landslide over time. Survey instrument M19 tracks displacement in this upper region and has been returning variable rates of deformation averaging 23 mm/year since filling of the Revelstoke Reservoir. Temporal differences in these rates are observed and these likely reflect individual blocks experiencing localized periods of activity and inactivity. Inclinometers S08 (Figure 4.6) and S51 (Figure 4.7); located at the same spatial position, but operated over different periods of time, are located near the down-slope boundary of
the upper region. These profiles show deformation through the lower shear zone, some internal deformation through the landslide mass and minor slip on a secondary shear zone.

Figure 4.5: LiDAR image of the upper region clearly shows hummocky terrain.
Figure 4.6: Inclinometer S08 shows deformation distributed across the lower shear zone and also through the landslide mass. Dark and light grey horizons (in all inclinometer figures) mark the basal and internal shear zones respectively, as recorded in borehole geology logs.

Figure 4.7: Inclinometer S51 shows deformation distributed across the lower shear zone, through the landslide mass and also within the secondary shear zone.
4.4.3 South Trough

The south trough, which runs approximately parallel to the side and head scarps, is a depression characterized by numerous internal scarps and tension cracks (Figure 4.8). The south trough has been interpreted to mark the south boundary of the main landslide mass (Patton and Hodge 1975, Kalenchuk et al 2009a). This interpretation corresponds well with findings summarized by Gerraghty and Lewis (1983) where drilling from the south adit, one of two drainage adits developed on site prior to reservoir filling, found an extremely fractured water bearing zone about 150 m north of the south scarp. Patton and Hodge (1975) interpreted from the south trough that total slide displacements are approximately 250 to 300 meters.
Figure 4.8: (top) Scarp feature within the south trough, (bottom) overgrown jumbled talus. Inset aerial photographs show locations where photographs were taken (modified after Kalenchuk et al. 2009a)
4.4.4 Talus Slopes

Between the south trough and the scarps is an area of jumbled talus. These talus slopes have accumulated from gradual ravelling of the scarps. Fresh ravelling as well as talus slopes overgrown with old growth forest are observed on site, demonstrating that continued ravelling has been ongoing for a long time.

4.4.5 Central Region

The central landslide region features gentle slopes overgrown by old growth forest. There is some differences in morphology between the north- and south-central regions (Figure 4.9). The south-central area is relatively featureless with mature topography. The relatively smooth nature of the area may suggest that this portion of the slope has remained intact or that there has been little continued deformation since initial landslide activation and the early landslide features have since been worn away. A boundary between the north- and south-central regions is marked by a subtle topographic break suggesting some degree of continued activity to the north where the terrain is slightly hummocky. Survey monuments M15 and M43 are located within the central region. These instruments show effectively negligible deformation (averaging 2.9 and 2.8 mm/year since reservoir filling). Inclinometer S23, located in the northern part of the central region, shows that deformation at this point on the landslide is dominated by surficial movement, with only minor slip on the basal slip surface and internal slide deformation (Figure 4.10). This inclinometer record appears to track deformation occurring below the basal slip surface. It is possible that this reflects deeper creep; the borehole geology log for S23 records jointing to be prominently parallel to foliation, these joints are very closely to moderately spaced and some are slightly sheared. Below 250 m
depth the rockmass is described as more competent with discrete slightly sheared horizons. Inclinometer S03 (Figure 4.11) shows deformation through the landslide mass, with some minor slip occurring on a shallow internal shear zone. This shallow shear is likely related to a minor extension of the active zone which is discussed in the following section.

Figure 4.9: LiDAR image of the central region shows varying morphology between the north- and south-central regions.
Figure 4.10: Inclinometer S23 shows surficial activity with minor displacements on the basal shear and insignificant movement through the landslide mass.

Figure 4.11: Inclinometer S03 shows slip on upper internal shear zone, with negligible deformation through landslide mass or along the lower internal shear surface. This inclinometer does not extend deep enough to intersect the basal shear zone.

The difference in rate of deformation between the upper region and the central region implies that there should be some zone of accumulation between these areas of the
landslide. Accumulation zones are generally characterized by evidence of thrusting, or bulging morphology; such a zone is not evident in the available data for this region, however more detailed site investigation may return more information. Otherwise, it may be speculated that there is no apparent zone of accumulation because the location of zones of accumulation and extension can vary over time (Picarrelli and Russo 2004). Early landslide activity may have initially extended only as far as the boundary between the upper and central regions, leaving a zone of depletion which has since transitioned to a zone of accumulation as retrogressive blocks simply "catch-up" with the lower landslide mass.

### 4.4.6 Lower Region

The lower region features much more irregular terrain than observed through the central region; depressions, crevices, internal scarps and fracture traces are more common (Figure 4.12). The south-lower area is a broad, over-steepened ridge marked by east-west trending lineaments roughly parallel to the south trough and south scarp. These features indicate north-south extension through the south-lower region, however without detailed mapping it is difficult to conclude pure extension or translational extension with some down slope shearing component. The slight clockwise rotation in lineament orientation at lower elevations and bulging of the toe does suggest some down slope directed deformation. Monuments M52 and M20 track deformation averaging 4.3 and 8.3 mm/year, respectively, since reservoir filling. At higher elevations, deformation primarily occurs through the landslide mass in the south-lower area (S02, Figure 4.13 and S12, Figure 4.14), and closer to the toe, discrete slip on the basal shear surface becomes more apparent (S14, Figure 4.15 and S30, Figure 4.16). Activity along an internal shear
is evident in S14, the noisy profile above this shear may indicate the upper portion of the landslide mass is discontinuous. S30 shows considerable surficial deformation associated with toe sloughing. Deformation below the basal slip surface are evident in S02 and S30. There is no geological record available for borehole S02 and borehole geology logs for S30 report the jointing at this depth to be closely to widely spaced and coated with clay and various minerals including pyrite, talc, calcite and quartz, there is no reported evidence of shearing.

Figure 4.12: LiDAR image of lower region. The broad ridge of the south-lower region shows east-west trending extensional lineaments, the north-lower region features the north and south lobes of the active zone, and toe sloughage is evident along the reservoir.
Figure 4.13: Inclinometer S02 shows shearing on the basal slip surface and some back-rotated deformation through the landslide mass.

Figure 4.14: Inclinometer S12 shows slightly back-rotated deformation through the landslide mass, with no displacements isolated along shear zones.
Figure 4.15: Inclinometer S14 shows shearing on the basal slip surface and some back-rotated deformation through the landslide mass. Noisy data above the internal shears may indicate a more disturbed portion of the slide profile.

Figure 4.16: Inclinometer S30 shows shearing on the basal slip surface as well as considerable surficial deformation.

The north portion of the lower region is the active landslide zone which is a depressional basin defined at the edges by scarp features and internally by hummocky, disturbed terrain. Two lobes are evident in LiDAR data where morphology features clearly define
areas of higher deformation rates (Figure 4.12). Survey monument M08 is located within the southern lobe of the active zone and measures movement rates averaging 19.2 mm/year since reservoir filling. Unfortunately there are no instruments located in the northern lobe to measure slope activity in this area. A number of inclinometers (Figures 4.17 to 4.22) are located within the active area, none of these instruments show well developed deformation along the lower shear surface; rather, deformation appears to be distributed throughout the slide mass. Deformation below the basal slip surface are evident in S43; however there is no geological record available for this borehole so evidence of shearing is unknown. The lower region moves faster than the middle region and the boundary between these areas marks a zone of depletion. This is particularly evident along the upslope boundary of the north-lower region where the landscape is marked by scarp features and sinkholes (Figure 4.23).

![Figure 4.17: Inclinometer S01 does not extend deep enough to provide a profile through the entire landslide mass, deformation though the upper portion of the mass is evident.](image)
Figure 4.18: Inclinometer S07 shows no slip on the lower shear and deformation through the landslide mass (according to 25/06/79 data only). Between 1979 and 1984 the borehole was blocked and more recent data provides deformation data above 130 m depth only.

Figure 4.19: Inclinometer S17 shows deformation through the landslide mass with no displacements specifically isolated along internal shear zones. This inclinometer does not extend deep enough to intersect the basal shear zone. Surficial deformation associated with toe sloughing is evident.
Figure 4.20: Inclinometer S43 shows minor slip on both the basal and internal shear surfaces, more so along the latter, with some deformation through the landslide mass, particularly below the internal shear.

Figure 4.21: Inclinometer S44 shows deformation through the landslide mass.
Figure 4.22: Inclinometer S47 does not extend deep enough to span the entire landslide profile. The available data shows surficial deformation associated with the toe slough region.

Figure 4.23: Photographs taken in the depletion zone at the boundary between the north-middle and north-lower regions. (top left) Valley parallel trough, (bottom left) internal scarp, (right) sink hole. Inset shows area where photographs were taken.

Along the reservoir, the lower region features extensive toe sloughing. The extent of this surficial instability is clearly evident in LiDAR imagery (Figure 4.12). Survey
monuments M47 (1.5 mm/yr), M48 (10.3 mm/yr), M41 (57.2 mm/yr), M57 (16.1 mm/yr) and M61 (12.8 mm/yr) all fall within this region and, with the exception of M47, return deformation rates higher than those monuments located just beyond the toe slough region. These high rates are primarily due to surficial movement rather than overall landslide behaviour. The toe slough region is also apparent in inclinometer data as instruments S30, S47, S17 and S47 all show well developed surficial movement. Back rotation is evident in S17 and S44, this reflects the rotational failure mode occurring at the landslide toe.

4.4.7 North Knob

The north knob is a prominent pinnacle of rock surrounded on all sides by extensional features and slopes covered in talus (Figure 4.24), giving the impression that material is failing away from this high point in all directions. This knob is likely a remnant morphological feature from early slide instability. Today, the north knob is inactive as indicated by survey monument M107 which records negligible deformation. Inclinometer S09 (Figure 4.25) is not installed deep enough to intersect the basal slip surface, however S09 data does show some offset towards the southeast likely reflecting rockmass relaxation towards the active area.
Figure 4.24: (upper) North knob viewed from the north, (middle) tension crack located southeast of the knob, and (lower) extensional feature observed near the boundary of the north knob and the basin (red arrows indicate direction of extension) (modified after Kalenchuk et al. 2009a). Inset shows location where photographs were taken.
Figure 4.25: Inclinometer S09 shows deformation through the landslide mass, directed southeast towards the active area and no displacements are specifically isolated along internal shear zones. This inclinometer does not extend deep enough to intersect the basal shear zone.

4.4.8 Over-Steepened Slopes

To the east and north of the north knob are over-steepened slopes (Figure 4.26). This region shows ongoing surficial deformation manifested in the field as curved tree trunks and surficial colluvium deposits. Morphological observations in this region show no prominent features to suggest modern deep seated displacements. Survey monument M11 records slope parallel movement averaging 19.6 mm/year since reservoir filling.
4.4.9 Toe Slump

An active toe slump bounded by scarp features (Figure 4.27) occurs near the landslide toe just north of the north-lower zone (Figure 4.26). Survey monument M62 returns anomalously high deformation rates (287.4 mm/yr average since reservoir filling) making this zone the most active area of Downie Slide. The high rates observed here are not representative of the overall slide behaviour because, as shown in inclinometer S32 (Figure 4.28), these movements are predominantly the result of surficial deformation. S32 shows possible deformation occurring below the basal slip surface. The obvious jump in surficial displacements between June 1984 and May 1986 is a response to toe inundation by reservoir filling. The geological log for this borehole reports alluvial deposits (approximately 6 m thick) directly below the basal slip surface, below here jointing ranges from very closely to widely spaced with occasional clay and mineral infillings and no reported evidence of shearing.
Figure 4.27: Photographs taken along the boundary of the active north toe area; (left) scarp exposure, (right) magnitude of offset at the side boundary of the active area (modified after Kalenchuk et al. 2009a). Inset shows photograph source area.

Figure 4.28: Inclinometer S32 shows significant surficial deformation in the toe slump region with minor displacement on the lower shear and negligible deformation through the landslide mass.
4.4.10 Basin

Upslope from the north knob is a basin region bounded by zones of depletion marked by extensional features including scarps and sinkholes. To the east and west of the basin is ridge and trough morphology (Figure 4.29); the west bounding ridges and troughs are particularly evident in topographic observations and LiDAR data (Figure 4.30). East-west trending extensional features mark the north boundary of the basin region (Figure 4.30). The basin itself slopes gently to the south, opening up, and draining into, the north-lower zone. It is hypothesized that material from this region has gradually displaced south-southeast towards the active area, however there are no survey monuments present to provide magnitude and direction of modern, local deformation. Monument M50, located at the north boundary of the basin, shows negligible movement since reservoir filling. Inclinometer S50 (Figure 4.31) is located within this region, and it shows a very slight (less than 70 mm over 14 years) south-southeast directed deformation through the landslide mass.
Figure 4.29: LiDAR image of the basin region highlighting the trough and ridge morphology as well as extension along the north basin boundary.
Figure 4.30: Photographs depicting extensional features at the north basin boundary, inset shows location where photographs were taken.
4.4.11 North Disturbed Zone

The north disturbed zone (Figure 4.32) is interpreted as a region of secondary failure that would have initiated in response to the main instability to the south. Localized movement is directed northeast, east and southeast towards the over-steepened slopes, basin and main slide mass, respectively. This region is hummocky and terraced; modern, active deformation is believed to be surficial, however this is difficult to conclude without sub-surface deformation data.
4.4.12 Lobe

The lobe (Figure 4.32) is interpreted to be a secondary failure. This region is bounded by two linear depressions, the upper portion of the lobe does not feature any morphology to suggest landslide activity; however, the lower portion is terraced giving the impression of shallow failure though glacial deposits or ground cover rather than deep seated failure through bedrock. This extension is likely initiated by the loss of material near the toe of the lobe due to gradual movement of the over-steepened slopes. Northeast directed surficial movement is active today as evidenced by curved tree trunks.
4.4.13 Slide Behaviour

Interpretations of overall landslide behaviour must consider observations of morphology as well as slope monitoring instrumentation. Morphological features are useful in recognizing the interaction between different landslide zones, for instance the recognition of zones of accumulation or depletion. Morphology can also provide some insight into the nature of slope deformation, for instance whether movements are deep-seated or surficial. The previous section looks at slide behaviour for each of the individual zones, and now an assessment of all zones together is used to interpret the overall modern landslide behaviour. Downie Slide is interpreted to be a massive, active complex, compound rockslide where the main landslide body can be divided into the upper, central, and lower regions, with secondary instabilities on the landslide flanks including the talus slopes, the north destabilized zone, the lobe and the basin, as well as the over-steepened slopes and the toe slough area. It is important to remember that these interpretations reflect the modern Downie Slide and that it is difficult, if not impossible, to conclude how the initial slide behaved when instability began during deglaciation.

As described in more detail above, failure mechanisms vary spatially across the slope. The basal shear zone primarily follows weak mica horizons and translational sliding occurs, or has occurred at some point in landslide history, through the central and upper regions of the slide. Today the upper region show evidence of retrogressive behaviour and the central region shows negligible displacement rates. In the lower landslide region the instability mechanism transitions from translational to rotational where the basal slip surface curves to outcrop in the valley; here, particularly in the south-lower region we see a transition from deformation distributed throughout the landslide mass to discrete slip on
the basal slip surface. Failure mechanisms also vary from north to south along the toe. The most northerly part of the landslide toe (over-steepened slopes and toe slough zones) has very minor slip on the basal shear surface, negligible deformation through the landslide and significant surficial activity (S32). Through the active central toe region most deformation occurs through the landslide mass (S17, S44), with significant surficial deformation in the toe slump area (S17, S47, S44). Sloughing along the reservoir acts to unload the landslide toe through erosional processes and may be an important control on slide behaviour, particularly in the active zone. In the south-lower region, shearing is evident on the basal slip surface, with lesser deformation through the landslide showing some back rotation.

The northwest areas of Downie Slide, including the north disturbed zone and the lobe, are interpreted to be secondary surficial instabilities likely developed in response to the larger unstable mass to the south and east. The talus slopes along the southern portion of Downie Slide were generated by gradual slope ravelling. The north knob is a morphological feature that is likely attributed to early slope instability. Currently this region is inactive and does not contribute to modern slope deformation. The over-steepened slopes and toe slump areas down slope from the north knob show surficial deformation.

When interpreting slide behaviour from instrumentation data it is important to consider the deformation processes occurring at each instrument location. Instruments located in areas where surficial deformation has been recognized will return elevated deformation rates that do not reflect the overall slide displacement rates. For example, the toe slough, toe slump and over-steepened slopes at Downie Slide have all been recognized as
surficial zones, and survey monuments in these areas return high deformation rates. Figure 4.33a demonstrates that when all survey monuments are considered on a contoured map of the overall slope behaviour there is an extremely fast zone near the northeast toe. This corresponds with monument M62, located on the active toe slump. By removing this point from the data set and reinterpreting overall slide behaviour it becomes apparent (Figure 4.33b) that the upper region of the slope and the active area in the central toe move relatively faster than the low deformation rates through the central portion of the slide. With recognition that a number of survey monuments are located within the toe slough zone and on the over-steepened slopes, instruments M41, M11, M47, M48, M57 and M61 can also be removed from the data set to create a slope deformation plot that reflects the true overall landslide behaviour (Figure 4.33c). Now, the active zone is still evident in the north-lower zone, and the highest deformation rates are observed near the head scarp where failure is retrogressive. The central portion of the slide shows very little, to negligible, deformation rates; as would be expected by the lack of morphological features observed in this area.
Figure 4.33: Contour plots of displacement rate standard deviation from the mean measured between 1990 and 2003; (a) all survey data utilized, (b) anomalous toe slump data is removed and (c) all data points with significant surficial deformation is removed to best demonstrate the true overall landslide behaviour.
In the analysis of overall slide behaviour some consideration has been given to the direction of deformation relative to the slope of topography (Table 4.1). Areas moving faster than 10 mm/year show fairly good correlation between displacement direction and slope aspect (<25° difference), with the exception of M19 which is located in the retrogressive failure region where individual blocks may be subject to rotation forces depending on the localized state of activity. Areas with the lowest deformation rates (effectively negligible displacements at less than 2.5 mm/year) return the poorest correlation between displacement direction and slope aspect, this is because the movement magnitude is less than the instrument error margin (about 5 mm) and these regions of the slope are considered to be effectively inactive. Correlation between slope aspect and direction of movement also corresponds with the morphological region. Poor correlation is observed in the upslope region, the north knob and the basin, for reason of localized rotation and negligible displacements. The best correlations are observed in areas interpreted to be affected by surficial deformation (M48, M61, M11, M41 and M62), and in the active zone (M08).

When drawing conclusions about observed slope behaviour, consideration must be given to the spatial distribution of monitoring instruments. For example, it has been concluded here that the upper retrogressive region exhibits some of the fastest observed deformation rates at Downie Slide, with the exception of surficial mechanisms near the landslide toe. This characterization of the upper region is based on a single data point (M19). Localized instabilities are anticipated for individual landslide blocks within the retrogressive area, and as such it is possible that M19 displacement rates may be anomalously low or high. In the absence of additional data for this region it must be assumed, based on observation
of morphological features, that this instrument is an adequate representation of the upper region. The design of any future additions to the instrumentation network at Downie Slide should take into consideration the installation of additional survey points in sparsely populated slope regions, particularly those areas interpreted to be contributing to the overall landslide behaviour. The analysis of Downie Slide would benefit from additional instruments through the upper region, and in the north lobe of the active zone. Areas with lesser contribution to the overall slide behaviour, for instance the lobe could be given lower priority for increased instrument density.

Table 4.1: Comparison of measured movement direction to local slope aspect. *Field rate based on average between 1990 and 1999. **Slope aspect is direction of slope dip at a survey monument location averaged over a 100 m x 100 m area.

<table>
<thead>
<tr>
<th>Monument</th>
<th>Field Rate* (mm/yr)</th>
<th>Movement Direction (°)</th>
<th>Slope Aspect* (°)</th>
<th>Difference (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M50</td>
<td>1.2</td>
<td>76</td>
<td>66</td>
<td>10</td>
</tr>
<tr>
<td>M47</td>
<td>1.2</td>
<td>21</td>
<td>66</td>
<td>-45</td>
</tr>
<tr>
<td>M43</td>
<td>1.3</td>
<td>80</td>
<td>81</td>
<td>-1</td>
</tr>
<tr>
<td>M107</td>
<td>1.5</td>
<td>91</td>
<td>48</td>
<td>43</td>
</tr>
<tr>
<td>M15</td>
<td>2.4</td>
<td>55</td>
<td>95</td>
<td>-40</td>
</tr>
<tr>
<td>M52</td>
<td>3.6</td>
<td>64</td>
<td>71</td>
<td>-7</td>
</tr>
<tr>
<td>M03</td>
<td>5.7</td>
<td>77</td>
<td>42</td>
<td>35</td>
</tr>
<tr>
<td>M20</td>
<td>7.9</td>
<td>61</td>
<td>54</td>
<td>7</td>
</tr>
<tr>
<td>M48</td>
<td>10.0</td>
<td>55</td>
<td>46</td>
<td>9</td>
</tr>
<tr>
<td>M61</td>
<td>12.7</td>
<td>57</td>
<td>56</td>
<td>1</td>
</tr>
<tr>
<td>M11</td>
<td>16.7</td>
<td>48</td>
<td>45</td>
<td>3</td>
</tr>
<tr>
<td>M57</td>
<td>19.1</td>
<td>80</td>
<td>50</td>
<td>30</td>
</tr>
<tr>
<td>M08</td>
<td>21.9</td>
<td>63</td>
<td>62</td>
<td>1</td>
</tr>
<tr>
<td>M41</td>
<td>56.9</td>
<td>60</td>
<td>56</td>
<td>4</td>
</tr>
<tr>
<td>M19</td>
<td>66.5</td>
<td>105</td>
<td>60</td>
<td>45</td>
</tr>
<tr>
<td>M62</td>
<td>283.3</td>
<td>99</td>
<td>82</td>
<td>17</td>
</tr>
</tbody>
</table>

Downie Slide slope deformation is spatially discriminated; different regions of the landslide mass exhibit variable magnitude and direction of displacement rates. This study demonstrates that an assessment of slope kinematics must take into consideration both observed deformation from monitoring data as well as an understanding of the site
morphology, the mechanics influencing individual landslide zones and the interaction between these zones. It should be noted that a complete analysis of landslide behaviour must also account for temporal changes in slope behaviour. Research is ongoing to assess how temporal changes in modern slope behaviour are related to ground water fluctuations at Downie Slide.

4.5 Numerical Modelling

Numerical simulation of the Downie Slide utilizes 3DEC (3-Dimensional Distinct Element Code) (Itasca Consulting Group, Inc. Minneapolis, Minnesota, 2003) to develop three-dimensional mixed continuum-discontinuum landslide models. Models presented in this paper have been developed following the methodology summarized by Kalenchuk et al. (2010 – Chapter 3). Readers are encouraged to consult that paper for a review of numerical modelling background and detailed summary of model development including aspects such as; mesh density and grid size sensitivity testing, the inclusion of water tables, the buttressing load imposed by the reservoir, as well as boundary conditions and in situ stresses.

Continuum materials, the landslide mass and intact material below the landslide are all assigned Mohr-Coulomb failure criterion. Material properties have been assigned to represent reasonable values for inter-bedded schist and gneiss. The landslide is considered to be weak, disturbed material characterized by a rockmass modulus equal to 500 MPa, Poisson's ratio equal to 0.25, friction equal to 34º, cohesion equal to 1 MPa and a tensile strength of 40 kPa. In situ material is considerably stronger and stiffer than the disturbed landslide; the rockmass modulus is equal to 10 GPa, Poisson's ratio equals 0.25, friction is equal to 46º and the density is 2700 kg/m³. Cohesion and tensile strength for
the in situ material are assigned values of 1 GPa and 100 MPa respectively. These unrealistically high values ensure that no failure occurs outside of the landslide boundaries; a reasonable condition given that the scope of this modelling is to study the behaviour of an existing landslide rather than the propagation of new instability. Both the in situ and disturbed materials are assigned zero dilatancy.

Joint elements defining the landslide shear surfaces are considered to be perfectly plastic and are controlled by the Coulomb-slip constitutive model (Itasca Consulting Group, Inc. 2003). Discontinuities are assigned a friction angle equal to 19°, cohesion equal to 400 kPa and a tensile strength of 50 kPa. Dilation of joints is assumed to be zero and base case shear and normal stiffness values are set at 50 and 100 MPa/m, respectively.

Figure 4.34: Schematic illustrating material properties utilized in 3DEC simulations (modified after Kalenchuk et al. 2009b).

4.5.1 Model Development

Kalenchuk et al. (2010 – Chapter 3) assess basal slip surface geometry and shear zone stiffness as factors influencing simulated landslide behaviour. They found that geometry is a key factor influencing slide mechanics and that spatial variation in stiffness as a function of shear zone thickness plays only a minor role. Building on their work, those
geometries and spatial stiffness distributions which best simulated observed deformation patterns are utilized here.

A massive landslide can be numerically defined as one discrete block sliding on the basal slip surface or as multiple blocks interacting along internal shear zones while collectively sliding on a basal slip surface. The first series of model runs explores if the slide movement is best represented as one mass with low elastic modulus or if the rigor of defining intermediate secondary shears will influence the mobility of specific landslide regions and improve the simulation of slope kinematics. At Downie it is likely that secondary shears developed during early instability, and additional internal features have developed ongoing, extremely slow deformation, therefore; some internal shears may be continuous across different morphological zones while others are related to the development of these individual areas. Therefore, secondary shears are assumed to be either (a) continuous across the entire landslide mass or, (b) related to landslide zoning and confined to specific morphological zones. Three sets of runs have been developed with different basal slip surface geometries using two continuous surfaces defined by a minimum curvature algorithm (Golden Software, Inc. 2002, Smith and Wessel 1990) and kriging (Cressie 1990, Davis 2002) and a stepped geometry defined by a minimum curvature algorithm. Shear zone stiffness values of \((K_s, K_n)\) equal to \((250 \text{ MPa/m}, 500 \text{ MPa/m})\), \((50 \text{ MPa/m}, 100 \text{ MPa/m})\) and \((10 \text{ MPa/m}, 20 \text{ MPa/m})\) are applied to thin, average and thick shear zone regions (Figure 4.35), respectively. A continuous minimum curvature algorithm is used to define all secondary shear zones based on the spatial prediction methodology described by Kalenchuk et al. (2009c – Chapter 2). Stiffness
parameters for secondary shears are held constant at $K_n = 100$ MPa/m and $K_s = 50$ MPa/m.

Figure 4.35: (left) Spatial variation in the thickness of the basal shear zone. (right) Thick, average and thin regions used to assign spatially varied stiffness parameters to shear surface in 3DEC simulations (Kalenchuk et al. 2010 – Chapter 3).

The second series of simulations gives consideration to the landslide morphology zones, assuming that the continuity of intermediate shears is influenced by these zones. The active zone at Downie Slide is an obvious starting point for isolating zone specific internal shears. Using borehole intercepts within this zone and the spatial extent of the active zone, as defined by morphological features, an intermediate shear is numerically simulated.

A second series of model runs tests the interaction between internal morphological zones. Here, the simulation of Downie Slide as a massive monolithic landslide is compared to various adjustments to the landslide boundaries to include or exclude specific landslide zones. These adjustments to landslide boundaries include: (1) a smoothed south boundary which follows the south trough, removing the talus slopes from the main landslide mass, (2) a smoothed north boundary which removes the lobe zone from the main landslide mass, (3) removal of the north knob and the over-steepened slopes, and (4)
removal of all secondary failure zones including the north disturbed zone, the lobe, the basin and the talus slopes, as well as the north knob area and over-steepened slopes (Figure 4.36). Using these adjusted landslide boundaries shear surface geometries have been developed using the best fit spatial prediction algorithms, including the minimum curvature algorithm and the multiquadratic radial basis function (Aguilar et al. 2005, Hardy 1990) based on cross validation and visual assessment procedure outlined by Kalenchuk et al. (2009c – Chapter 2).
Figure 4.36: Comparison of various adjusted landslide boundaries to include or exclude specific landslide zones to create differing monolithic representations; (a) smoothed south (b) smoothed north (c) no knob (d) hypothesis. (left) Aerial photograph with dark long-hatch line showing boundary of monolithic mass and white short-hatch line showing adjusted geometry, (right) cross sections where solid grey line shows basal shear surface for the adjusted geometry and hatched line shows monolithic mass.
4.5.2 Comparison of Field and Simulated Data

Field observations are compared to numerical simulations quantitatively using $R^2$ values, and qualitatively by visual assessment of spatial patterns projected on contoured plots of displacement rate standard deviation from the mean (Figure 4.37). Field data provides movement rates in mm per year at the location of survey monuments. Numerical models return displacement rates in terms of mm per time step at the same geographical locations on the landslide surface. By standardizing rates to the average plus or minus standard deviation about the global average it is possible to directly compare real and simulated slide behaviour.

Numerical simulations are further compared to field data by direction cosine values ($c_i$) calculated using the north and east components of displacement measured at all instrument points ($i$) in both field ($e_{fi}, n_{fi}$) and model ($e_{mi}, n_{mi}$) data sets:

$$c_i = \frac{(e_{fi} \times e_{mi} + n_{fi} \times n_{mi})}{\sqrt{(e_{fi}^2 + n_{fi}^2)} \times \sqrt{(e_{mi}^2 + n_{mi}^2)}}$$ (4.1)

When deformation sampled at a specific point in a numerical model is parallel to the equivalent field point, the cosine value is equal to one; for perpendicular deformation the value is equal to zero.
Figure 4.37: Two models with stepped minimum curvature basal slip surface geometry and varying interpretations of internal secondary shears are used as examples to illustrate the comparison of field and modelled data. (top) Displacement rate standard deviation from the mean measured in the field vs. modelled returns $R^2$ values used to quantitatively compare various model simulations (bottom) Contoured displacement rate standard deviations for visual comparison.
4.6 Results and Discussion

4.6.1 Intermediate Shear Zones

The first series of models test how the inclusion of secondary failure zones influences slope kinematics. Figure 4.38 illustrates the contoured relative displacement rates. These results demonstrate that inclusion of secondary intermediate shears can improve the simulation of slope behaviour observed at Downie Slide. Based quantitatively on $R^2$ and qualitatively on visual inspection of slope deformation patterns, the stepped minimum curvature basal shear geometry with the inclusion of two continuous internal shears, or one internal shear confined to the active zone, produce the best results, returning $R^2$ values of 0.90 and 0.88 respectively. The continuous minimum curvature basal shear geometry with inclusion of two continuous secondary shears also does well to reproduce the spatial pattern observed in field data.

Cosine plots (Figure 4.39) demonstrate that the inclusion of internal shears has negligible influence on the direction of deformation observed across the landslide mass. Comparisons between modelled and observed direction of deformation is favorable for all instruments with the exception of M19. Poor directional correlation in the upper portion of the slope is attributed to localized deformation mechanisms not accounted for in the continuum landslide mass. Retrogressive failure occurs in the upper region and large rock blocks are subject to differential displacements and rotations.
Figure 4.38: Contoured deformation rate standard deviation about the mean measured in numerical models at the location of field survey monuments for varying basal shear surface geometries defined by (left) continuous minimum curvature algorithm, (center) continuous kriging function (right) discontinuous minimum curvature algorithm.
Figure 4.39: Cosine values depicting the correlation in direction of deformation between field and simulated data points for varying basal shear surface geometries.
Limitations to the improvement of simulated slope behaviour by the inclusion of secondary shears may be related to a number of factors. First, inclinometer data shows that internal shears do not discretely contribute to a significant portion of the observed slope deformation. Therefore, it is not anticipated that a reasonable representation of secondary shears in a numerical simulation would drastically change the modelled behaviour. Further, it is possible that secondary shears within a landslide mass may or may not interact with each other. It is known that Downie Slide is disturbed, highly fractured and containing multiple internal shears, and it is therefore reasonable to conclude that the application of a realistically low elastic modulus to the landslide mass sufficiently accounts for all internal slope deformation.

4.6.2 Boundary Adjustments

Analysis of the morphological features and slope behaviour lead to the hypothesis that the Downie Slide does not behave as a monolithic slide mass, but rather as a composite rockslide with numerous morphological zones which exhibit variable failure mechanisms and behavioural patterns. The next series of models makes a number of adjustments to the landslide geometry in order to explore how the kinematics of this massive compound slide are influenced by the interaction between morphological zones. Results for the adjusted boundary models are summarized in Figures 4.40 and 4.41.

Smoothing the south boundary to remove the talus slopes from the simulated mass produces very poor results with $R^2$ values of 0.04 and 0.16 for the exclusion and inclusion of a secondary shear defining the active zone, respectively. This poor correlation with field data is attributed to the upper portion of the simulated mass moving considerably slower, relative to the middle and lower region than is observed in the field and achieved
in earlier simulations of the full landslide extent depicted in Figure 4.38. This relative slowing of the upper region may in part be due to the removal of the talus slopes which likely contribute some loading to the main landslide body.

From this suite of model runs the smoothed north boundary produces the best results, returning $R^2$ values of 0.67 and 0.80 for the exclusion and inclusion of the active zone secondary shear, respectively. These results are comparable to simulations of the entire landslide area. Given the minor influence on displacement patterns, it is concluded that the lobe region has negligible influence on the overall slide behaviour.

Removal of the north knob region returns moderate results ($R^2 = 0.57$) when the active zone is excluded and poor results ($R^2 = 0.07$) with inclusion of the active zone secondary shear. These simulations return deformation rates at M50 which are quite low relative to the remaining slide mass. These slow rates upslope from the north knob region are the result of geometric hang-ups associated with curving the boundary around the west and south edges of the north knob. In all adjusted boundary simulations, morphological boundaries are assumed to be continuous and discrete physical boundaries and the landslide masses are surrounded by the stronger, and stiffer in situ material. This effectively creates hard boundaries at the slide edges. Hard contacts, while realistic at the external slide boundaries, are not, however, realistic at internal zone boundaries where, in reality, neighbouring material is comprised of disturbed landslide debris.

Simulation of only the main slide body achieves poor results for both the exclusion and inclusion of the active zone secondary shears. Where the active zone is excluded, the upper region moves too slowly relative to the rest of the slide mass, this is attributed to the lack of loading by the adjacent talus zone. The poor correlation for the inclusion of
the active zone in the adjusted north knob boundary and main slide body models are attributed to the toe region achieving high displacement rates relative to the rest of the landslide mass, which exceed those observed in the field and in simulation of the full landslide extents.

Adjustments to the landslide boundary have little influence on the overall slide direction of deformation. Cosine plots for adjusted boundaries show negligible differences to those plotted for models simulating the full extent of the landslide mass (Figure 4.41), indicating that changes to the slide boundaries have more control on the rate of deformation than on the directional component.

These results clearly demonstrate that, with the exception of the lobe region, the interactions between zones play a significant role in overall slide behaviour. Neglecting these interactions has resulted in inadequately simulated slide behaviour. During early instability the entire slide area may have been moving as a monolithic mass. Since early instability the modern morphology zones have evolved and they continue to mechanically interact along internal boundaries rather than behaving independently.
Figure 4.40: Contoured deformation rates measured in numerical models at the location of field survey monuments for various adjustments to landslide boundary geometries.
Figure 4.41: Cosine values depicting the correlation in direction of deformation between field and simulated data points for various adjustments to landslide boundary geometries.
4.7 Summary

Downie Slide has been interpreted as a massive, composite rockslide. Slope deformation is spatially discriminated where varying magnitude and direction of displacement rates have been observed across different regions of the landslide mass. A number of landslide zones have been defined based on interpretation of morphological features and a detailed assessment of spatial variation in slope behaviour. The main landslide mass, made up of the upper, central and lower regions, features translational, rotational and retrogressive mechanisms. An active zone is found within the north-lower region and a toe slough area extends along the reservoir shoreline. Secondary instabilities surround the main landslides, including the talus slopes, the disturbed north zone, the lobe, the basin, the over-steepened slopes and the toe slump. The north knob is an inactive area, with negligible contribution to modern observations of overall landslide behaviour.

Complex three-dimensional numerical modelling has demonstrated that slide behaviour is influenced by secondary shears within the landslide mass and the mechanical interaction between landslide zones. The inclusion of secondary shears within the landslide mass provided some improvement in the simulation of landslide behaviour. However, detailed models may be difficult, if not impossible, to develop without considerable site investigation, first, to recognize morphological zones within a massive landslide and, second, to have sufficient data to define the geometry of internal shear zones. Representation of the landslide mass using a low deformation modulus can achieve reasonable results by allowing internal strains to be adequately accommodated by deformation within the simulated landslide. The influence of secondary shears should be
tested on a case-by-case basis as the role of internal structure varies for specific case studies.

Adjustments to landslide boundaries are influenced by the simulation of hard boundaries and loading from secondary zones. Modelling adjusted boundaries based on morphological regions has assumed continuous and discrete physical boundaries between the landslide masses and the stronger, and stiffer in situ material. This effectively creates hard boundaries at the slide edges, which is a realistic contact at the external slide boundaries. However these types of boundaries are not realistic at internal zone boundaries where, in reality, neighbouring material is comprised of disturbed landslide debris. Further, in the adjusted boundary simulations there is no load from adjacent landslide zones acting on the simulated landslide mass, however in reality the talus slopes, the destabilized north zone and the basin would generate some load acting on the main slide mass.

Understanding the physical nature and observing the ongoing behaviour of slow moving massive slope movement allows researchers to interpret key components influencing slope deformation and these interpretations can then be tested using three-dimensional numerical models. Assessments of massive landslide kinematics must consider both observed deformation from monitoring data as well as an understanding of the site morphology, the mechanics influencing individual landslide zones and the interaction between these zones. It is concluded here that the secondary shears at Downie Slide play a minor role in the overall slope mechanics which are primarily controlled by the geometry of the basal slip surface, and secondary shears should only be included where justified by morphological zoning, for example through the Downie Slide active area.
Further, the interaction between zones plays an important role in overall slide behaviour and as such the full extent of landslide mass must be modelled in order to achieve reasonable simulation of observed slope deformation.

4.8 Acknowledgements

The authors would like to thank BC Hydro, particularly John Psutka and Dennis Moore, for site and data access. This work has been made possible through contributions by NSERC, CFI and GEOIDE.

4.9 References


Kalenchuk, K.S., Diederichs, M.S. and Hutchinson, D.J., 2010. Three-dimensional numerical simulations of the Downie Slide to test the influence of shear surface geometry and heterogeneous shear zone stiffness. Submitted to Computational Geosciences manuscript # COM368. 24 manuscript pages.


CHAPTER 5*

Downie Slide - Sensitivity of Water Table Interpretations to Temporal Changes in Data Distribution

5.1 Abstract

Ground water monitoring at Downie Slide began in 1973. Since then, various piezometers have been decommissioned and new instruments, at different locations, have been installed. Over the operating life of a consistent set of borehole piezometers, interpolated ground water fluctuations reflect measured changes in ground water levels. When the distribution of boreholes changes, apparent ground water fluctuations are observed at close proximity to decommissioned or installed instruments. Unless these apparent fluctuations coincide with an adjustment to ground water boundary conditions, they are probably artifacts of the changing data set. This paper explores the sensitivity of ground water interpretations to instrument decommissioning and installation, and

* This Chapter has been submitted for publication as:
proposes data extrapolation in order to avoid apparent, and incorrect, interpretations of water table fluctuations.

5.2 Introduction

Downie Slide is a massive \((1.5 \times 10^9 \text{ m}^3)\) active, composite, extremely slow moving rockslide located 64 km north of the Revelstoke Dam on the west bank of the Revelstoke Reservoir, in the Columbia River Valley, British Columbia, Canada (Figure 5.1). The landslide is found within a metasedimentary sequence which overlays the Monashee Core Complex (Read and Brown 1981, Scammel and Brown 1990 and references therein refs, Armstrong et al. 1991, Johnston et al. 2000) and is truncated to the east by the Columbia River Fault Zone (Figure 5.1). The highly fractured rockmass is locally composed of inter-layered schist, gneiss and quartzite (Imrie et al. 1991), with a well developed mica foliation dipping down-slope towards the east. Shear zones predominantly follow these weak mica rich horizons.

The instability was first recognized in 1956 during reconnaissance mapping for a possible dam site. The British Columbia Hydro and Power Authority recognized the potential hazard to future hydro-electrical developments created by slides and initiated an intensive exploration program. Commencing in 1973, the program involved geological mapping, collecting ground water data, monitoring slope movements, and locating sub-surface shear zones by drilling and seismic methods (Moore et al. 1997). Continued piezometric monitoring provides an impressive record of ground water conditions over the operating life of the Revelstoke Reservoir. This study of Downie Slide ground water conditions takes data from twenty boreholes (Figure 5.1). Multiple piezometric tips located at varying elevations in each borehole provide level records for multiple water tables.
Piezometric data is collected, on average, once every two weeks from April to October, and no data is collected over winter months.

Changes to ground water boundary conditions have occurred since reservoir development and over the operating life. Prior to reservoir filling extensive drainage development was completed between 1974 and 1982, and involved two adits (Figure 5.1) totalling 2.43 km with more than 13.6 km of boreholes. Reservoir filling took place between October, 1983 and August, 1984. Since the time of reservoir filling, gradual drainage system deterioration has resulted from borehole shearing and infilling of silty material and bio-accumulation (Imrie et al. 1991, Enegren and Imrie 1996). Drainage system maintenance is a major component of continued hazard management at Downie Slide. Drainage system upgrades were completed in 2008 to re-establish maximized slope drainage, however given the timing of this research the influence of this recent maintenance has not been considered.

By interpreting ground water levels annually, it is possible to explore how these changes to boundary conditions influenced ground water levels over the reservoir operating life. This paper demonstrates how periodic ground water interpretations are sensitive to the decommissioning and installation of piezometers and how data extrapolation is necessary to avoid apparent, and incorrect, interpretations of water table fluctuations. It is important to note that this interpretation of ground water conditions is not meant to be a detailed hydrogeological analysis. Landslide hydrogeology is notably complex, with localized low- and high-permeability rockmass regions influenced by the character of fracture networks, faulting or clay rich shears and rockmass dilatancy controlled by compressional or extensional states of activity. This study aims to develop reasonable water table states
that can be applied to numerical models for simulating slope response to changing water table boundary conditions (Kalenchuk et al. 2010 – Chapter 6).

![Map of Downie Slide](image)

**Legend**
- Columbia River
- Piezometers
- Metasediments
- Slide Boundary
- Granodiorite
- Drainage Adits
- Pluton
- Fault Zone

**Figure 5.1:** Downie Slide map illustrating local geological setting, topography and location of borehole piezometers and drainage adits (adit portals are at approx. 590 m a.s.l.), (inset) location map of the Downie Slide in southeastern British Columbia, Canada.

### 5.3 Water Table Interpolation

Multiple water tables have been interpreted within the Downie Slide; constrained by low permeability of the basal slip surface and secondary shears within the landslide mass.
One water table is below the basal slip surface, one between the basal and a secondary slip surface and a possible third one above secondary shears (Figure 5.2). These water tables vary in piezometric head, ground water chemistry (Bourne and Imrie 1981) and the magnitude of response to seasonal precipitation and changing boundary conditions controlled by reservoir operations.

Figure 5.2: Cross-section AA’ illustrating multiple water tables identified at Downie Slide; they are confined by the low permeability of landslide shear surfaces.

Ground water interpretations for massive landslides are often based on sparse borehole piezometers, relying largely on interpolation and extrapolation to define water tables for all extents of a slope. Spatial interpretation of ground water conditions can be completed statistically using a number of algorithms for predicting spatial patterns (Kalenchuk et al. 2009 – Chapter 2). Water tables at Downie Slide have been interpolated using a minimum curvature algorithm (Golden Software Inc. 2002, Smith and Wessel 1990). This algorithm has been selected based on minimum error during cross-validation tests and visual assessment of spatial patterns as suggested by Kalenchuk et al. (2009 – Chapter 2). It is important to use a consistent spatial prediction algorithm when
comparing ground water conditions at different periods in time, mixing spatial prediction techniques will result in apparent ground water changes.

5.3.1 Artifacts of Changing Data Sets

Interpretation of ground water levels is sensitive to the spatial and temporal variations in data density. Spatial variations occur in irregularly spaced data sets. Interpolations of sparsely sampled regions may be over-smoothed compared to regions with denser data clusters. Temporal variations in data density, the focus of this paper, occurs when the pattern of data collection changes over time, a common phenomena for long term monitoring programs, as old instruments are decommissioned and new ones are installed. At Downie Slide, piezometric installation programs took place in 1974, 1976-1977, 1981 and 1993 (Figure 5.3). Intermittent piezometer decommissioning has also occurred as boreholes are sheared or become blocked, and when instruments malfunction. When ground water levels are interpreted periodically, temporal variations in data density can create incorrect artifacts. Figure 5.4 illustrates apparent fluctuations of the middle water table between 1992 and 1993. These are recognized as incorrect artifacts of changing data distribution as there are no changes in ground water boundary conditions over this period that would induce significant water table fluctuations. The middle water table will be used for illustrative purposes throughout this paper; data trends recorded for the lower water table at Downie Slide are similar.
Figure 5.3: Piezometer installation and decommission record, shaded regions show operating life of instrument in each borehole.

Figure 5.4: In 1993 a number of instruments were installed in boreholes S49, S48, S54, and S52 creating apparent changes in the ground water levels. These apparent changes in the level of the middle water table between 1992 and 1993 are artifacts of changes to data distribution.
5.3.2 Extrapolation of Piezometric Records

Figure 5.3 illustrates the operation of piezometers at Downie Slide, when each instrument is either brought on-line or taken off-line apparent ground water fluctuations similar to those illustrated in Figure 5.4 occur when comparing successive annual water table levels. To correct these apparent, and incorrect, ground water fluctuations piezometric data can be extrapolated through time. To do this, it is assumed that if no ground water boundary conditions change (i.e. drainage development or reservoir filling) ground water trends can be extrapolated backwards or forwards in time to the last/next change in boundary conditions that would influence ground water proximal to the instrument being extrapolated.

Confirmation that an area has been influenced by changing boundary conditions can be achieved by looking at neighbouring instruments. For example, instruments in boreholes S45 and S52 are installed in 1992 and 1993 respectively. The installation of these instruments creates an apparent drop in ground water levels through the central portion of the landslide. Data from surrounding boreholes S31, S36, S37 and S38 (Figure 5.5, see Figure 5.4 for borehole locations) demonstrate that there has been no change in ground water boundary conditions in this area since the drainage system was built (this area is not influenced by reservoir filling). Therefore, the S52 and S45 data can be extrapolated backwards by linear trendline to improve data density in previous years as early as 1978 the year where drainage development was completed in close proximity to these holes.
Figure 5.5: Data from instrumented boreholes S31, S36, S37 and S38 demonstrate that there has been no change in ground water boundary conditions in the area proximal to S45 and S52 since the drainage system was built. Therefore it is reasonable to extrapolate S45 and S52 data backwards in time.

Another example of apparent ground water changes occurs in 2001 when S13 is taken offline. The removal of this instrument from the data sets creates an apparent drop in the ground water levels in the north portion of the slope between 2001 and 2002. S13 data trends up to 2001 (Figure 5.6), and those trends in neighboring borehole (S38, S45/S25/S49), show a gradual increase over time likely due to loss of drainage capacity, however there is no substantial jump in ground water levels over this period. It is
therefore reasonable to assume there has been no abrupt change in boundary conditions and S13 data can be extrapolated forward by linear trendline to more recent years.

Figure 5.6: (top-left) Apparent change in ground water levels from 2001 to 2002 as S13 is taken offline, (top-right) change from 2001 to 2002 when S13 data is extrapolated forward. (bottom) Piezometric trends for boreholes near S13 show no change in boundary conditions after 2001; therefore it is reasonable to assume that S13 data can be extrapolated forward in time.
These examples, S13, S45 and S52, have been extrapolated based on linear trendlines. In some cases, more complex extrapolation can be achieved through more rigorous assessment of changing ground water conditions. For example, consider boreholes S25 and S49, which have the same spatial location, but operate during different periods of time. An apparent water table rise occurs in 1979 when S25 P2 (sampling the middle water table) goes offline, this is not considered to be reasonable as there is likely a substantial post-drainage drop in this instrument that there is no record for. S25 P1 (sampling the lower water table) did not malfunction, providing a continuous data record between 1977 and 2003. S25 P1 shows significant response to drainage development in 1976-1977 and 1982-1983, indicating that S25 P2 would have similar drawdown. Also, S49 P1 correlates well with S25 P1 between 1993 and 2003. Given these similarities it can be assumed that using the S25 P1 trendline data can be extrapolated between 1978 and 1993 to fill in the missing data between S25 P2 and S49 P2. Boreholes S25 and 49 are too far upslope to be influenced by reservoir filling (as shown by neighboring instruments), therefore S49 P2 is back extrapolated to 1983 using the trendline defined by

\[ Y_{S49P2',i} = -0.0815t^2 + 326.29t - 325853 \]  

(5.1)

Which is derived from S25 P1

\[ Y_{trendline} = -0.0815t^2 + 326.29t - 325890 \]  

(5.2)

It is obvious that S25 P2/S49 P2 is influenced by drainage, and the drainage response is extrapolated based on the S25 P1 drainage response defined by


(5.3)
Figure 5.7 illustrates the S25/49_{\text{middle}} extrapolation. Figures 5.8 illustrate how extrapolation of this instrument influence the apparent change in ground water levels between 1992 and 1993.

Figure 5.7: Extrapolation of S25/49 borehole piezometers to attain a complete middle water table record at the spatial location of these instruments.

Figure 5.8: Installation of borehole S49 influence of data interpretation between 1992 and 1993; (left) interpreted from raw data (right) interpreted from piezometric data where S49 data has been extrapolated through time.
5.4 Results and Discussion

Figure 5.9 compares interpreted ground water fluctuations for non-extrapolated and extrapolated data sets with changing boundary conditions at each stage of reservoir operations, including: drainage development, reservoir filling, and reduced drainage capacity. With no data extrapolation, there appears to be very little influence of drainage on the region directly upslope from the Adit 1, which is unlikely, given the magnitude of response observed in operating instruments such as at those installed in boreholes S38 (-34 m observed for middle water table) and S25 (-96 m observed for lower water table). With extrapolated data, significant drawdown is interpreted through the central portion of the slide (up to approximately 150 m), due to installation of drainage measures. There is no direct evidence of actual drawdown in this area of the landslide, so this magnitude of drawdown may be slightly high, given that the largest observed response to drainage is -96 m. There is little variation between the ground water response to reservoir filling interpreted from raw data versus extrapolated data, as no instruments were decommissioned or installed during this period.

Data extrapolation does significantly influence interpretation of capacity losses in the Downie Slide drainage infrastructure. The installation of piezometers in the early 1990s lowers the interpreted water tables, creating an apparent drop in ground water levels over the operating life of the reservoir. This drop is not believed to be realistic, as most operating instruments actually show gradual increases in water tables. Using extrapolated data the gradual rise in water table is evident; however caution must be taken when assessing the magnitude of this interpreted rise as it is dependent on extrapolated data.
BC Hydro has been successful in their mandate to more than offset the influence of reservoir filling by development of the slope drainage system. Drainage development was effective in lowering deformation rates in the landslide mass with the most significant response observed throughout the lower region of the main landslide body (Kalenchuk et al. 2010 – Chapter 6). The main landslide body showed negligible response to reservoir filling, while accelerations were observed in secondary, surficial instabilities along the inundated toe. Over the operational life of the Revelstoke Reservoir, the overall landslide behaviour has been steady-state as changes in deformation rates have been effectively negligible. Slope stability analyses by Enegren (1995) have concluded that the slope factor of safety is more favorable, even with modern losses in drainage capacity than it was prior to drainage development. Analysis of monitoring data by Kalenchuk et al. (2010 – Chapter 6) demonstrates that slope deformation rates are lower than those observed pre-drainage. These studies demonstrate the continued adequacy of the drainage infrastructure.
Figure 5.9: Ground water fluctuations with changing boundary conditions at each stage of reservoir operations, including; (top) drainage development, (middle) reservoir filling, and (bottom) reduced drainage capacity; interpreted from raw data (left), and from piezometric data which has been extrapolated through time (right).

5.5 Summary

Changes in ground water levels over time can be assessed by comparing annual water table interpretations. Misinterpretations of these periodic comparisons may result from temporal variations in data density. For instance, at Downie Slide, piezometric
installation programs took place in 1974, 1976-1977, 1981 and 1993, and instruments have been intermittently decommissioned. These installation and decommissioning cycles have created temporal changes in the spatial data distribution, leading to the interpretation of apparent, and incorrect, water table rises and falls. Extrapolation, based on a key assumption that ground water trends are constant for periods where there are no changes to boundary conditions, has been done to remove incorrect data artifacts from the ground water analysis.

The extrapolation of ground water data through time has improved interpretations of water table changes at Downie Slide over the operating life of the Revelstoke Reservoir. Drainage development lowered ground water levels significantly in slide areas proximal to adits and boreholes, while portions of the slope more distal to drainage infrastructure saw no significant changes. Ground water response to reservoir filling was observed only by piezometers in close proximity to the inundated landslide toe. In the 28 years since reservoir filling, small increases in piezometric levels have been observed in portions of the landslide near drainage infrastructure. These small increases are likely related to gradual deterioration of the drainage system due to borehole shearing, and infilling of silty material and bio-accumulation.

This paper has demonstrated the sensitivity of ground water interpretations to temporal variation in data density due to the installation and decommission of individual instruments over time. The approach to temporal data extrapolation improves the understanding of changing ground water levels over time. Differences between the interpretations of raw and extrapolated ground water data sets are most significant to the assessment of drainage system efficiency. Drawdown, in response to drainage
development, is larger than would be interpreted based on raw data interpretation; indicating that drainage is more effective than may otherwise be concluded. Further, losses to the drainage capacity are more evident as artifacts in the raw data set allude to a misinterpreted fall in ground water levels over the reservoir operating life.

5.6 Acknowledgements

This work has been made possible through contributions by NSERC, CFI and GEOIDE. Thanks also to BC Hydro, particularly J. Psutka and D. Moore, for site and data access.

5.7 References


Golden, Colorado, USA: pp 89-162


CHAPTER 6*

Downie Slide – Numerical Simulation of Ground Water Fluctuations Influencing Behaviour of a Massive Landslide

6.1 Abstract

Ground water levels at Downie Slide have varied during the development and over the operating life of the Revelstoke Reservoir. Drainage system construction successfully lowered ground water levels through the central portion of the slope; reservoir filling resulted in water table rise near the inundated toe, and over the operating life of the hydro-electric facility gradual, minor losses to the drainage system capacity have resulted in a slow rise in water table levels. Calibrated models capable of reproducing observed deformation patterns at Downie Slide have been tested with changing ground water levels. Models perform well, adequately reproducing observed global slope response to changes in piezometric boundary conditions. These models have also been used to

* This Chapter has been submitted for publication as:
Kalenchuk K.S., Hutchinson, D.J. and Diederichs, M.S., 2010. Downie Slide - numerical simulation of ground water fluctuations influencing behaviour of a massive landslide. Submitted to Landslides manuscript # LASL 243. 26 manuscript pages
forward test potential trigger scenarios, including rapid reservoir drawdown and a total loss of drainage system capacity.

6.2 Introduction

Ground water is a common variable influencing deformation patterns observed in large active landslides; for example Beauregard (Miller et al. 2008, Barla et al. 2009), Vaiont (Semenza and Ghirotti, 2000, Hendron and Patton 1987), Ruinon (Crosta and Agliardi 2003), Áknes (Nordvik and Nyrnes 2009) and La Clapière (Rat 1995, Cappa et al. 2003) to name a few. When continuous deformation is slow, an understanding of how changing ground water levels influence temporal slope behaviour is very important to hazard management. Due to the magnitude of kinetic energy associated with massive landslides, the most realistic approach to hazard management is often to live with them, acknowledging potential trigger scenarios, and mitigate, where economically reasonable, using techniques such as slope drainage. However, given the physical scale and complexity of massive landslides, it is often difficult to refine and proof remediation techniques, and to forward test potential trigger scenarios.

Numerical simulations can be applied to improve hazard management and cost-benefit analyses of mitigation techniques by training models to simulate slope deformation. Once simulated behaviour can adequately reproduce slope deformation observed from field monitoring, trained models can be used to explore how deformation rates may be slowed, or eliminated, to test the effectiveness of mitigation techniques. Further, simulations can be done to forward test trigger scenarios as a means to assess the magnitude event required to incur significant slope accelerations.
Downie Slide has been monitored for over 35 years, and data shows that the modern slope behaviour is largely controlled by changing ground water boundary conditions. Prior to reservoir filling, the British Columbia Hydro and Power authority (BC Hydro) invested in an extensive slope drainage system that aimed to more than offset the anticipated water table rise associated with inundation of the landslide toe. Drainage development took place in stages between 1974 and 1982. It involved two adits totalling 2.43 km and a network of more than 13.6 km of boreholes. Drainage resulted in significantly lowered water levels through the central portion of the slide and achieved reduction in slide deformation rates. Reservoir filling occurred between October 1983 and August 1984. During this time, water level increases were only observed in close proximity to the inundated toe, where some accelerated deformation was observed, but these are believed to be associated with surficial failures near the reservoir and are not thought to be reflective of overall landslide behaviour. Over the operating life of the hydroelectric project, drainage system deterioration has lead to gradual increases in water levels through the central portion of the slide. These increases have been minor and deformation rates have remained relatively constant.

Extensive three-dimensional numerical modelling of the Downie Slide has been completed by Kalenchuk et al. (2010a – Chapter 3, 2010b – Chapter 4), exploring the influence of three-dimensional shear surface geometry, spatial variation in shear zone properties, the influence of internal shears and interaction between landslide zones. Building on these works, further modelling has been completed to numerically simulate temporal changes in slope behaviour that have been observed over the operating life of the Revelstoke Reservoir. Using trained models, testing has been done to consider the
effectiveness of drainage infrastructure, and assess future possible trigger scenarios to explore the impact of rapid reservoir drawdown and continuing loss of drainage capacity. This numerical modelling does not produce specific values, such as, deformation rates at a given water table level, as this was not the research goal. Rather, it improves the understanding of landslide processes, particularly changes in slope kinematics as a response to introduced perturbations.

6.3 Case Study

Downie Slide is a massive, active, composite, extremely slowly moving rockslide located on the west bank of the Revelstoke Reservoir in the Columbia River Valley, approximately 64 km north of the Revelstoke Dam, in British Columbia, Canada (Figure 6.1). The estimated 1.5 billion cubic meter landslide is composed of inter-layered schist, gneiss and quartzite (Imrie et al. 1991). The highly fractured rockmass is dominated by a gently inclined joint set (22° dip and 080° dip direction) sub-parallel to the local foliation. In addition, there are two sub-vertical joint sets that dip roughly east and north. The geological setting has been studied in depth by Brown and Psutka (1980), Jory (1974), Wheeler (1965), and others; readers are referred to these works for detailed explanation of the local and regional geological and structural setting.

A number of morphological regions (Figure 6.1) have been interpreted by Kalenchuk et al. (2010b – Chapter 4) from aerial photographs, LiDAR, site visits and slope monitoring data, and a detailed assessment of landslide morphology and spatially discriminated slope behaviour and failure mechanism has been completed. The main landslide mass includes the upper, central and lower regions of the slope, and features translational, rotational and retrogressive failure mechanisms. An active zone occurs within the north-lower region,
and sloughing is ongoing at the landslide toe along the reservoir shoreline. Secondary instabilities are found to the south and north of the main landslide mass, including the talus slopes, the disturbed north zone, the lobe, the basin, the over-steepened slopes and the toe slump. An inactive area referred to as the north knob makes negligible contribution to modern observations of overall landslide behaviour.

The basal slip surface at Downie Slide dips shallowly towards the river valley, roughly sub-parallel to the topographic surface. At the landslide toe, this shear zone rotates to outcrop near the valley floor. The basal slip surface is a sheared zone that ranges in thickness from less than 2 to nearly 50 m, and additional secondary shears are found within the landslide mass. Shears are characterized as zones of foliation parallel joints, closely to very closely spaced, with areas of fractured and crushed material, clay and mica gouge and chlorite altered clay (Bourne et al. 1978, Bourne and Imrie 1981, Gerraghty and Lewis 1983).
Figure 6.1: (top left) Location map of the Downie Slide in the Columbia River Valley in southeastern British Columbia, (top right) local geological setting and (bottom) aerial photograph of the Downie Slide showing landslide zones based on landslide morphology and spatially discriminated slope behaviour (modified after Kalenchuk et al. 2010a – Chapter 3).
6.4  **Ground water Conditions**

The ground water setting at Downie comprises multiple water tables confined by low permeability shear zones. This includes a lower water table confined below the basal slip surface, an upper water table confined within the slide and a possible perched water table near the landslide toe (Figure 6.2). The lower and upper water tables will be the focus of this study, as there is sufficient data to characterize these two piezometric surfaces. These two water tables vary by piezometric head, geochemistry (Bourne and Imrie 1981), and the magnitude of response to changing ground water boundary conditions and seasonal fluctuation.

![Figure 6.2: Schematic of a typical cross-section illustrating multiple water tables that have been identified at Downie Slide (Kalenchuk et al. 2009a). For numerical simulation, the lower water table (confined below the basal slip surface) is applied to the basal slip surface and material below the basal slip surface; the upper water table is applied to material within the landslide and internal secondary shears.](image)

**6.4.1 Interpreting Water Tables from Piezometric Data**

Piezometric data provides ground water levels at specific points in space at specific points in time. In order to define water tables for the full three-dimensional extent of the
landslide over the operating life of the Revelstoke Reservoir spatial and temporal interpolation and extrapolation of data is required.

**Spatial Considerations**

At any point in time, the three-dimensional water table state can be interpreted based on piezometric data and key ground water assumptions. For Downie Slide it is assumed that water tables at the toe of the slide are related to either the river level or the reservoir elevation, depending on the period of interest (pre- or post-reservoir filling). Fully three-dimensional water tables are interpreted using a rigorous procedure incorporating both spatial statistics and engineering judgment; this procedure is based on the spatial prediction techniques described by Kalenchuk et al. (2009b – Chapter 2). Spatial prediction algorithms are applied to the available piezometric data, and the river or reservoir elevation, to generate ground water surfaces across the landslide area. A number of algorithms have been tested and, in this case a minimum curvature algorithm (Golden Software Inc. 2002, Smith and Wessel 1990) has been selected base on minimum error returned through a cross-validation procedure and visual assessment of the trueness of the resulting piezometric surface based on knowledge of the local ground water conditions.

**Temporal Considerations**

Temporal data extrapolation is required for interpreting ground water levels over the Revelstoke Reservoir operating life due to changing data density over time. Piezometric installation programs took place at Downie Slide in 1974, 1976-1977, 1981 and 1993, and instrument decommissioning has occurred as boreholes have been sheared or became
blocked, or when instruments have malfunctioned. Kalenchuk et al. (2010c – Chapter 5) discuss in detail how incorrect ground water interpretations can result from temporal differences in data density and provides a methodology for data extrapolation through time. Temporal data extrapolation is based on a key assumption that ground water trends are constant for periods where there are no changes to boundary conditions.

6.4.2 Temporal Variation in Ground water Conditions

The ground water setting at Downie Slide has been significantly impacted by the operation of the Revelstoke Reservoir. Ground water levels have fluctuated over time in response to changing boundary conditions resulting from the development of drainage infrastructure (1974-1982), toe inundation during reservoir filling (1983-1984) and gradual losses in drainage system capacity over the reservoir operating life (1985-2003, note that 2003 is the limit for data availability during this research, the reservoir continues to operate and monitoring is ongoing). Using spatial and temporal data interpolation and extrapolation, water tables have been interpreted for the Downie Slide to establish a number of ground water states including pre-drainage, post-drainage, post-reservoir filling, and late-reservoir life (Figure 6.3).

Studies dating back to 1965 investigated the potential influence of the Revelstoke Reservoir on Downie Slide stability. These studies, particularly studies from 1973 to 1976 under the guidance of the Downie Slide Panel of Experts, resulted in the development of two adits totalling more than 2,430 m with more than 13,600 m of drainholes (Bourne and Imrie 1981). Landslide drainage aimed to improve slope stability by reducing uplift pressures on the basal slip surface (Enegren and Imrie 1996); mathematical modelling indicated that the reduction of ground water levels by localized
Drainage along the toe would offset the influence of reservoir filling (Enegren 1995). Drainage development occurred over three phases between 1974 and 1981 (Bourne and Imrie 1981), and ground water levels were successfully lowered through the central landslide mass, with little influence at higher elevations (Figure 6.4).

Figure 6.3: Cross-sections showing variations in water tables interpreted for different ground water states at Downie Slide, including pre-drainage, post-drainage, post-reservoir filling and late-reservoir life.
Figure 6.4: Water table changes (vertical m) between 1973 and 1983, due to drainage system development; interpreted using temporal data extrapolation (Kalenchuk et al. 2010c – Chapter 5); (top) middle water table, (bottom) lower water table.
Reservoir filling occurred between October, 1983 and August, 1984, submerging the toe of Downie Slide by more than 70 m at full reservoir capacity (573 m a.s.l. water elev.). During reservoir filling, only those piezometers near the inundated toe measured ground water rise and negligible change occurred at higher elevations (Figure 6.5). Since reservoir filling there have been very small increases in ground water levels due to gradual deterioration of the drainage system (Figure 6.6). This deterioration results from boreholes becoming blocked by shearing or infilling of silt material, and bacterial growth near borehole collars. In the early 1990s, borehole flushing was done in an attempt to regain some drainage capacity, however no long-term improvements were achieved.
Figure 6.5: Water table changes (vertical m) between 1983 and 1985 due to reservoir filling; interpreted using temporal data extrapolation (Kalenchuk et al. 2010c – Chapter 5); (top) middle water table, (bottom) lower water table.
Figure 6.6: Water table changes (vertical m) between 1985 and 2003 due to gradual losses in drainage system capacity over the Revelstoke Reservoir operating life; interpreted using temporal data extrapolation (Kalenchuk et al. 2010c – Chapter 5); (top) middle water table, (bottom) lower water table.
6.4.3 Seasonal Variation in Groundwater Conditions

Seasonal groundwater fluctuations show peak levels during summer months between May and August. In the spring and early summer, water levels rise rapidly and the peak is reached earlier in holes at lower elevations. Piezometric data is not collected over the winter, however based on observed water level drops through the autumn, it is inferred that minimum annual groundwater levels would occur during the winter months (January-April).

Figure 6.7 illustrates the spatial variation in the recorded magnitude of seasonal fluctuations, interpreted as a minimum variation, since no winter data is recorded. Upper regions of the slope experience higher seasonal fluctuations, typically in the order of 10 m, but in some cases exceeding 25-30 m and on rare occasions up to 85 m of change have been observed. In the middle region of the slide, fluctuations are closer to 5 m in magnitude, and near the toe fluctuations are typically in the order of 1 m. The cause of this spatial variation is two-fold. First, fluctuation magnitude likely reflects the effects of infiltration. The highly fractured upper region of the landslide is a recharge zone; here the dilated rockmass would allow for significant surface water infiltration. As a result, no surface streams are observed through the upper region. Through the central and lower regions of the slide, infiltration is limited by thick ground cover, and as such streams occur throughout the middle and lower regions. The second control on fluctuation magnitude is the proximity to the reservoir, as the water tables within the rockmass near the toe are most likely controlled by relatively steady state reservoir conditions and by the influence of drainage measures.
The influence of seasonal fluctuations on slope deformation rates have been considered for Downie Slide, unfortunately, the frequency of slope monitoring data (which is measured annually) is inadequate to assess the seasonal changes in landslide deformation rates. It is probable that higher rates occur during the summer months when water tables are elevated, with lower rates occurring during periods of low water levels.

Figure 6.7: Spatial variation in the minimum magnitude of seasonal ground water fluctuations at Downie Slide (upper water table). Data from point A shows high seasonal fluctuations near the head scarp recharge zone, and data from point B shows very little annual variation (Kalenchuk et al. 2009a). The lower water table shows similar trends.
6.5  Slide Behaviour

This paper focuses on the temporal changes in deformation rates at Downie Slide in response to changing ground water boundary conditions. Changes to slide displacement rates, interpreted from survey monument data, in response to drainage development, reservoir filling and gradual losses in drainage capacity over the operating life of the Revelstoke Reservoir, are illustrated in Figure 6.8. Table 6.1 summarizes the displacement rates over each period. Accelerations and decelerations less than ± 5 mm/year and annual displacement rates less than 5 mm/year are considered negligible, as such small magnitudes fall within the error margin of the measured data (instrument data below these 5 mm tolerances are noisy and shows no clear trends). Surficial instabilities, which are not representative of the global landslide behaviour, have been recognized near the landslide toe in the over-steepened slopes, toe slump and toe slough regions (Kalenchuk et al. 2010b – Chapter 4). The responses of these surficial failures to changing ground water boundary conditions vary considerably from the response of the main landslide body. While surficial behaviour does not contribute to global activity, it is important to recognize, and account for, these discrepancies. Therefore, for demonstrative purposes, Figure 6.8 illustrates plots with all data on the left, and with surficial data removed on the right.

Drainage development clearly shows decelerations across the entire landslide mass. Through the central and lower portion of the slide this is a direct response to lowered ground water levels. Decelerated rates measured through the upper region of the landslide (M19) are attributed to localized behaviour in of large blocks attributed to the retrogressive development of this upper region, rather than a direct response to drainage
development. The most significant response to drainage development is observed throughout the lower region of the main landslide body. Most instruments located along the slide toe, particularly those in surficial deformation zones show negligible changes in response to drainage development. Shallow instabilities are less influenced by water table drawdown because the water tables occur at depth below these surficial features and so pore pressures are not directly influenced by these areas of the rockmass.

Table 6.1: Summary of displacement rates (mm/yr) as measured by survey monuments.

<table>
<thead>
<tr>
<th>Monument ID</th>
<th>Pre-Drainage</th>
<th>Post-Drainage</th>
<th>Post-Reservoir Filling</th>
<th>Late-Reservoir Life</th>
</tr>
</thead>
<tbody>
<tr>
<td>M03</td>
<td>10</td>
<td>2</td>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td>M08</td>
<td>34</td>
<td>20</td>
<td>12</td>
<td>21</td>
</tr>
<tr>
<td>M11</td>
<td>7</td>
<td>26</td>
<td>20</td>
<td></td>
</tr>
<tr>
<td>M15</td>
<td>10</td>
<td>7</td>
<td>2</td>
<td>4</td>
</tr>
<tr>
<td>M19</td>
<td>19</td>
<td>1</td>
<td>9</td>
<td>31</td>
</tr>
<tr>
<td>M20</td>
<td>12</td>
<td>6</td>
<td>5</td>
<td>10</td>
</tr>
<tr>
<td>M41</td>
<td>52</td>
<td>54</td>
<td>59</td>
<td></td>
</tr>
<tr>
<td>M43</td>
<td>15</td>
<td>4</td>
<td>12</td>
<td>4</td>
</tr>
<tr>
<td>M47</td>
<td>3</td>
<td></td>
<td></td>
<td>2</td>
</tr>
<tr>
<td>M48</td>
<td>19</td>
<td>4</td>
<td>11</td>
<td>11</td>
</tr>
<tr>
<td>M50</td>
<td>14</td>
<td>8</td>
<td>5</td>
<td>2</td>
</tr>
<tr>
<td>M52</td>
<td>13</td>
<td>5</td>
<td>5</td>
<td>6</td>
</tr>
<tr>
<td>M57</td>
<td>36</td>
<td>14</td>
<td>15</td>
<td></td>
</tr>
<tr>
<td>M61</td>
<td>15</td>
<td>15</td>
<td>12</td>
<td></td>
</tr>
<tr>
<td>M62</td>
<td>10</td>
<td>287</td>
<td>223</td>
<td></td>
</tr>
<tr>
<td>M107</td>
<td>2</td>
<td>3</td>
<td>3</td>
<td></td>
</tr>
</tbody>
</table>
Figure 6.8: Deformation rates vary spatially and temporally, as different zones of the landslide show variable response to changing ground water boundary conditions (left) all instruments, and (right) surficial data is removed.

The slope response to reservoir filling is spatially discriminated; secondary surficial instabilities show different response than the main landslide body, and behaviour of the retrogressive upper slope again varies from the lower portions of the landslide. Instruments M11, M62, and M48 located within the over-steepened slopes, the toe slump and toe slough regions respectively, show considerable accelerations. The rise in piezometric levels through the northern landslide toe causes localized acceleration which is not reflective of the overall landslide behaviour. There are negligible changes through
the central and lower portion of the slope with some minor decelerations through the
central area of the main landslide body. These localized low magnitude decelerations are
not interpreted to be in response to reservoir filling as this area of the slope is sufficiently
far from the reservoir toe. The upper region of the landslide does show some minor
acceleration (M19 and M43), these are interpreted as localized variation in activity rather
than a direct response to reservoir filling. This area of the landslide is most influenced by
infiltration and localized variation in activity may be related to precipitation and snow
melt, however meteoric data has not been available for this analysis.

Over the operational life of the Revelstoke Reservoir, changes in landslide behaviour
have been effectively negligible. Change in activity occurs through the upper slope
region, again this is attributed to localized behaviour in the retrogressive upper region.
Minor accelerations are observed through the central portion of the lower region (the
active zone), this is a probably in response to gradual losses on drainage capacity. M11
shows minor decelerations which may indicate the surficial deformation of the over-
steepened slopes have settled down, following accelerated response to initial toe
inundation. With the exception of these minor, localized changes in deformation rate, the
overall landslide behaviour has been steady-state over the operating life of the Revelstoke
Reservoir.

Through reservoir filling and over the operating life of the Revelstoke Reservoir,
deformation rates have remained lower than those observed prior to drainage
development, with the exception of localized observations at M19. Therefore, even with
minor losses to drainage capacity, the mandate by BC Hydro to more than offset the
influence of reservoir filling by slope drainage continues to be achieved. Maintenance to
the drainage system is fundamental to continued hazard management; in 2008 drainage system upgrades were completed to re-establish maximized slope drainage, however given the timing of this research, the influence of recent drainage system maintenance has not been considered.

6.6 Numerical Modelling

Three-dimensional numerical models have been developed using 3DEC (3-Dimensional Distinct Element Code) (Itasca Consulting Group, Inc. Minneapolis, Minnesota, 2003) following the methodology for simulating massive landslides presented by Kalenchuk et al. (2010a – Chapter 3). A mixed continuum-discontinuum method is used to represent the landslide, and undisturbed in situ material, using deformable continuum blocks discretized by tetrahedral-shaped finite difference zones. These continuum blocks interact along geologically realistic shear surface geometries defined by discrete joint elements (Figure 6.9).

Boundary conditions include zero velocity boundaries in the direction normal to the model faces on the bottom and vertical edges of the model. The vertical to horizontal stress ratio is assumed to be equal to one, and in-situ stresses are initialized by assigning zone stresses as a function of rock density, depth below the topographic surface and gravity. Also, to account for the buttressing effect of the reservoir, a toe load is applied to the inundated portion of the slope.
Two water tables are accounted for in the numerical simulations. The lower water table is applied to in situ material below the landslide mass and to the basal slip surface, and the
upper water table is applied to material within the landslide mass and to internal shears. Pore-pressures are calculated by linearly interpolating the depth of water based on the local elevation of the appropriate piezometric surface. FISH (a programming language embedded within 3DEC) routines are used to apply specific pore pressures to all gridpoints (for continuum materials) and sub-contacts (for discontinuum elements).

Sophisticated three-dimensional numerical modelling by Kalenchuk et al. (2010a – Chapter 3, 2010b – Chapter 4) has demonstrated that slide behaviour is influenced by three-dimensional shear surface geometry, spatial variation in shear zone properties, secondary shears within the landslide mass and the mechanical interaction between landslide zones. These previous works have developed calibrated numerical models capable of simulating slope deformation patterns observed by field monitoring data. A calibrated model is now used to simulate temporal changes in slope behaviour. The model applied in this study simulates Downie Slide with the active zone (Figure 6.1) discretely defined by internal secondary shears using basal shear surface geometries defined by a stepped geometry interpreted using a minimum curvature algorithm.

The Mohr-Coulomb failure criterion is applied to continuum materials. The intact, in situ material below the landslide mass has been assigned rockmass modulus of 10 GPa, Poisson's ratio of 0.25, friction equal to 46° and 2700 kg/m³ density; these parameters reasonably represent the local geology; inter-bedded schist and gneiss. The in situ material is assigned high cohesion (1 GPa) and tensile strength (100 MPa) in order to avoid failure through the rockmass outside of the landslide boundaries. This is a reasonable control, as the focus of this modelling is the behaviour of an existing landslide rather than the propagation of new instability. The disturbed landslide material is
considered to be substantially weaker and less stiff than the in situ rockmass. It is characterized by rockmass modulus of 500 MPa, Poisson's ratio of 0.25, friction equal to 34°, cohesion equal to 1 MPa and tensile strength of 400 kPa. Both the in situ and disturbed materials are assigned zero dilatancy.

Landslide shear surfaces are represented as discrete discontinuities controlled by a perfectly plastic Coulomb-slip constitutive model (Itasca Consulting Group, Inc. 2003). Joint strength is defined by friction equal to 19°, cohesion equal to 400 kPa and a tensile strength of 50 kPa. The joint dilation angle is assumed to be zero, and joint shear and normal stiffness parameters vary spatially as a function of shear zone thickness based on the finding by Kalenchuk et al. (2010a – Chapter 3), the applied stiffness values are summarized in Figure 6.10.
6.6.1 Numerical Procedure

Models are run through a set-up sequence with pre-drainage ground water conditions.

This sequence (Table 6.2), applied by Kalenchuk et al. (2010a – Chapter 3), steps through
a number of stages designed to avoid shocking the models while achieving initial
equilibrium. Equilibrium is defined as the ratio of maximum unbalanced force in the
model to the maximum zone force (where zone force is the average zone stress multiplied
by area) equal to less than 1%. At the end of each stage gridpoint displacements and
velocities are initialized to zero.

Following the initial set-up sequence two series of runs are complete to test model
response to changing ground water conditions. The first series of runs examines changing
ground water boundary conditions over the reservoir operating life. Pre-drainage water
tables, applied during the set-up sequence, are maintained for a user specified period of
20,000 time steps such that steady state deformation rates are achieved. After 20,000
time steps the ground water levels are either held constant at pre-drainage levels or
adjusted to post-drainage, post-reservoir filling or late-reservoir life levels and cycled for
an additional 20,000 time steps. The rate of movement is measured over the second
10,000 steps of the later 20,000 in order to assure that steady state deformation rates have
been re-established.
Table 6.2: Summary of staged set-up to minimize the initial model deformation (Kalenchuk et al 2010a – Chapter 3).

<table>
<thead>
<tr>
<th>Stage</th>
<th>Material</th>
<th>Constitutive Model</th>
<th>Rockmass Modulus (GPa)</th>
<th>Poisson's Ratio</th>
<th>Density (kg/m³)</th>
<th>Cohesion (MPa)</th>
<th>Tensile Strength (MPa)</th>
<th>Friction (°)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>In situ and landslide</td>
<td>Linear-elastic</td>
<td>100</td>
<td>0.49</td>
<td>2700</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>No shear zone is implemented. Grid point pore pressures and in situ stress are initialized. High elastic properties prevent large deformation. High Poisson's Ratio allows lateral deformation of elements during the redistribution of initial stress concentrations.</td>
</tr>
<tr>
<td>B1</td>
<td>In situ and landslide</td>
<td>Linear-elastic</td>
<td>100</td>
<td>0.35</td>
<td>2700</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>The elastic properties of the rockmass are reduced, there is no shear implemented.</td>
</tr>
<tr>
<td>B2</td>
<td>In situ and landslide</td>
<td>Linear-elastic</td>
<td>10</td>
<td>0.25</td>
<td>2700</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Elastic properties of the rockmass are reduced to realistic value. There is no shear implemented.</td>
</tr>
<tr>
<td>C</td>
<td>In situ and landslide</td>
<td>Elastic-plastic, Mohr-Coulomb</td>
<td>10</td>
<td>0.25</td>
<td>2700</td>
<td>1000</td>
<td>100</td>
<td>46</td>
<td>The shear zones are implemented. High rock cohesion is applied to avoid plastic deformation and significant changes in the position of gridpoints.</td>
</tr>
<tr>
<td>C2</td>
<td>In situ and landslide</td>
<td>Elastic-plastic, Mohr-Coulomb</td>
<td>10</td>
<td>0.25</td>
<td>2700</td>
<td>1000</td>
<td>100</td>
<td>46</td>
<td>Sub-contact pore pressure is applied to shear surfaces.</td>
</tr>
<tr>
<td>D1</td>
<td>In situ</td>
<td>Elastic-plastic, Mohr-Coulomb</td>
<td>10</td>
<td>0.25</td>
<td>2700</td>
<td>1000</td>
<td>100</td>
<td>46</td>
<td>Material properties for the landslide mass are adjusted to approach more realistic values.</td>
</tr>
<tr>
<td></td>
<td>Landslide</td>
<td>Elastic, plastic, Mohr-Coulomb</td>
<td>1</td>
<td>0.25</td>
<td>2700</td>
<td>1</td>
<td>0.05</td>
<td>38</td>
<td></td>
</tr>
<tr>
<td>D2</td>
<td>In situ</td>
<td>Elastic, plastic, Mohr-Coulomb</td>
<td>10</td>
<td>0.25</td>
<td>2700</td>
<td>1000</td>
<td>100</td>
<td>46</td>
<td>The material properties of the landslide mass are reduced to realistic values.</td>
</tr>
<tr>
<td></td>
<td>Landslide</td>
<td>Elastic, plastic, Mohr-Coulomb</td>
<td>0.5</td>
<td>0.25</td>
<td>2700</td>
<td>1</td>
<td>0.04</td>
<td>34</td>
<td></td>
</tr>
</tbody>
</table>
A second series of model runs has been developed to test slope response to two potential trigger scenarios. The water tables that have been assumed are sudden reservoir drawdown and total loss in drainage capacity (Figure 6.11). The sudden reservoir drawdown scenario uses the late-reservoir life water table and removes the toe buttressing load, this assumes that following rapid drawdown, pore pressures within the slope remain high and require some period of time to dissipate. A water table representing total loss in drainage capacity is interpreted using pre-drainage piezometer data tied into the reservoir elevation at the landslide toe. For this second series of models, the initial set-up sequence is run using late-reservoir life water table conditions. Following the set-up sequence, the initial ground water state is maintained and the model is cycled for 20,000 time steps, ground water levels are then adjusted for an additional 20,000 time steps and deformation rates are, again, compared for the later 10,000 steps.

Figure 6.11: Cross-sections showing water tables interpreted to represent (top) rapid reservoir drawdown where late-reservoir life is applied with no buttressing load of the reservoir at the landslide toe and (bottom) total loss in drainage capacity under full reservoir operating conditions.
6.6.2 Comparison of Field and Simulated Data

It is not possible to directly compare field deformation rates (mm/yr) with numerically modelled deformation rates (mm/time step) and there is no logical correlation between numerical time steps and real time. Therefore, temporal changes in slope behaviour are compared using a normalized change in rate approach. Field data is normalized to 5 mm/year, as rates less than this magnitude are considered negligible and fall within the error margin of the measured data. Model data is normalized to 0.05mm/10000 time steps; a magnitude that has been selected to represent the tolerance for significant change in simulated deformation rate. Slope deformation is tracked in numerical models at the same geographical location as those field instruments which are representative of global slope behaviour.

6.7 Results and Discussion

Figure 6.12 illustrates the normalized change in rate data for field observations and numerical simulations through different stages of the Revelstoke Reservoir development and operation. Negative and positive normalized values indicate decelerations and accelerations respectively. Values between -1 and 1 are indicative of negligible change in rate. This numerical modelling has not aimed to reproduce specific, observed values; rather the goal of this research has been to improve understanding of how composite landslide processes respond to changing ground water boundary conditions and to demonstrate the need for sophisticated numerical models for simulating these complex changes in slope kinematics as a response to introduced perturbations.
Figure 6.12: Modelled and measured normalized change in deformation rates through sequential changes to ground water levels over Revelstoke Reservoir development and operation.

Based on qualitative comparison, the simulated slope response to drainage system development corresponds reasonably well with observed change in deformation rates. All data points in the model run show decelerations, or negligible change, however the magnitude of the normalized change in rate is lower for the modelled data than for the measured data. The difference between normalized response observed in modelled and measured data is in part related to the normalizing process, as 5mm/yr does not correspond directly to 0.05 mm/time step. The difference in magnitude may be also be
related to complexities in the hydro-mechanical coupling in the natural slope that is not numerically accounted for. In reality, a change to ground water level will influence a number of additional factors; for example: hydrostatic pressures in the rockmass, fracture networks and on the shear surfaces, as well as rockmass density and dilatancy. In these numerical models, changes to ground water levels only influence pore pressure in the rockmass and acting on shear surfaces.

The modelled response to reservoir filling has poor correlation with field observations. Models show accelerations through the lower and central portions of the slope, indicating that simulated global behaviour is more sensitive to localized water table rises during toe inundation. This heightened sensitivity is in part attributed to the fact that the simulated landslide mass is a continuum and therefore middle slope regions are more influenced by dynamic conditions occurring at the toe. In reality, the fractured and disturbed rockmass would have natural disconnects between the lower and central portions of the slope. Accelerations observed in the field through the upper portion of the slope are not reproduced by numerical simulation which shows negligible change in this area. The discrepancies at high elevations reflect the interpretation that the upper slope is not influenced by ground water changes, as deformation is the result of localized activity related to retrogressive behaviour and the simulated continuum landslide material reproduces global deformation patterns rather than complex, localized activity.

With gradual, minor rises in water table levels over the reservoir operating life, the modelled and observed slope behaviour both show, for the most part, negligible changes in global slide behaviour. Both the modelled and measured data produce minor accelerations in the central portion of the slope; this is a direct response to increased
piezometric levels due to losses in drainage capacity. Once again localized activity is
evident in the field observations of the upper slope, and these are not numerically
replicated.

The simulated response to hypothesized water table changes; rapid reservoir drawdown
and total losses in drainage capacity are illustrated in Figure 6.13. Rapid reservoir
drawdown has negligible influence on the global landslide behaviour. In reality it is
likely that surficial instabilities near the landslide toe; particularly the toe slump and toe
slough regions would have a more significant response than the main landslide body, as
these shallow instabilities would be more sensitive to the loss of buttressing load while
pore pressures remain elevated. If this ground water scenario were to trigger accelerated
toe failure, the compounded influence of high water tables and rapid unloading of the toe
may induce some global response, particularly through the lower landslide regions and
the active zone. The simulated loss of drainage capacity results in small accelerations
through the lower portion of the slope in the same order of magnitude as decelerations
observed in numerical simulation of drainage system development. It is therefore
concluded that in the absence of drainage, deformation rates would likely be similar to
those observed in pre-drainage field data, and the influence of a full reservoir would not
induce any significant accelerations in global slope behaviour.
Through the development, and over the operating life of the Revelstoke Reservoir, the Downie Slide has exhibited spatially discriminated responses to changing ground water levels. This variability results from complexities in the landslide setting, for example complex shear surface geometry, spatial variation in material strength parameters, influence of internal secondary shears and the mechanical interaction between landslide zones. Also, the magnitude of change to ground water levels is not constant across the full extent of the slope creating a non-uniform change in stress state along slip surfaces and through the landslide mass. Adding to the complex response is the fact that changes
to ground water levels occur over a period of time: drainage development was completed in stages between 1974 and 1982, the reservoir was filled between 1983 and 1984, and gradual losses in drainage capacity during reservoir operation have spanned 18 years of available data (1985-2003). The rise and fall in ground water levels would be gradual, and localized change would be controlled by rockmass permeability and infiltration rates and complicated by localized factors such as shears and fracture networks.

The modelling results presented here demonstrate that through numerical simulation is it possible to achieve reasonable, based on qualitative assessment, reproduction of temporal changes in slope behaviour in response to changing ground water boundary conditions. The best correlation between modelled and measured change in slope behaviour are for the simulations of drainage development and the gradual losses in drainage system capacity. These changes to ground water boundary conditions have demonstrated a more global influence over slope behaviour. Reservoir filling had a more localized influence on the inundated toe which was not numerically reproduced. Accelerations observed in numerical data through the central slope region during reservoir filling are attributed to the continuum nature of numerical models compared to the fractured and disturbed nature of the real slide mass. Another significant discrepancy has been identified in the upper slope, where localized deformation is not related to changing ground water levels.

Instances of poor correlation between reality and models can also be attributed to the inherent complexities of the natural massive landslide; heterogeneous rockmass quality, variable geology and structure, and non-uniform fluctuations in ground water levels. It is important to note that dynamic water levels are not simulated and changes to ground water conditions are instantaneous in the numerical models. Improvements to simulated
behaviour in numerical models may be achieved with the application of staged or gradual changes in ground water levels. Without extensive and detailed geological and structural mapping, and hydrogeological investigations, it is impossible to further refine these already sophisticated numerical models. Given the scale of the Downie Slide, and the very low probability of catastrophic failure, it is not feasible to expect that such a high level of detail could be achieved.

6.8 Summary

Analysis of slope monitoring data concludes that landslide ground water is an important boundary condition influencing the modern behaviour of Downie Slide. From the analysis of temporal changes in landslide behaviour discussed in this paper, it is clear that different regions of the slope respond differently to changing boundary conditions. This reflects the fact that the modern Downie Slide is a massive, active, composite, extremely slow rockslide and localized failure mechanisms are occurring in different morphological zones. Retrogressive behaviour in the upper slope does not show any direct response to changing ground water levels and fluctuation in deformation rates through this region are interpreted to reflect localized activity. The central and lower portion of the slope show decelerations in response to drainage development, no significant response to reservoir filling, and minor accelerations due to gradual losses in drainage capacity. Surficial and secondary failures near the toe show negligible response to drainage system development and over the operating life of the reservoir. These surficial features exhibited a more significant response to reservoir filling as accelerations were observed through the over-steepened slope, toe slump and toe slough zones.
Numerical simulation of changing ground water conditions produces reasonable correlations with field observations, particularly for those changes to ground water boundary conditions that influence the global activity of Downie Slide. Simulated drawdown by drainage development slowed overall landslide deformation rates. Reservoir filling resulted in accelerations through the lower portion of the slope. Gradual losses in reservoir capacity over the reservoir operating life had negligible influence on models of global behaviour with the exception of minor accelerations near the toe throughout the active region. Modelled change in displacement rates in response to drainage development and gradual losses in drainage capacity reflect field observations reasonably well, poorer correlation was returned by the reservoir filling model.

Forward testing of potential trigger scenarios; rapid reservoir drawdown and complete loss of drainage capacity has demonstrated the application of these sophisticated numerical simulations to hazard management. Simulation of rapid reservoir drawdown resulted in negligible change to the global landslide behaviour, however in interpreting these results one must recognize that drawdown is likely to have more significant impact on surficial instabilities at the landslide toe, and the compounded influence of drawdown and accelerated toe unloading has not been accounted for in this study. Total loss in drainage capacity has resulted in accelerations of the same magnitude as the decelerations observed during simulated drainage development. While the total failure of the drainage system is extremely unlikely, particularly with continued site maintenance, this worst case scenario would likely only cause minor slope accelerations.

Analyses of piezometric and deformation monitoring data from active, massive composite, slopes can significantly improve the understanding of how complex
kinematics respond to changing ground water boundary conditions. With this knowledge base it is possible to numerically simulate observed changes in slope behaviour and to recognize, and account for, both consistencies and discrepancies between modelled and measured deformation. Understanding why, and where, numerical simulations correlate with or deviate from field observations enables experts to forward-test potential trigger scenarios, as demonstrated in this study, or to design and test the effectiveness of engineered mitigation approaches to controlled slope behaviour.

6.9 Acknowledgments

The authors would like to thank BC Hydro, particularly John Psutka and Dennis Moore, for site and data access. This work has been made possible through contributions by NSERC, CFI and GEOIDE.

6.10 References


Kalenchuk K.S., Diederichs, M.S. and Hutchinson, D.J., 2010a. Three-dimensional numerical simulations of the Downie Slide to test the influence of shear surface geometry and heterogeneous shear zone stiffness. Submitted to Computational Geosciences manuscript # COM368: 24 manuscript pages.

Kalenchuk K.S., Hutchinson, D.J. and Diederichs, M.S., 2010b. Morphological and geomechanical analysis of the Downie Slide using 3-dimensional numerical models:
testing the influence of internal shears and interaction between landslide regions on simulated slope behaviour Submitted to Landslides manuscript # LASL 241: 32 manuscript pages.


CHAPTER 7*

Integration of Three Sources of Slope Monitoring Data to Characterize Landslide Behaviour, Beauregard Landslide, Northwestern Italy

7.1 Abstract

Slope displacement monitoring at Beauregard Landslide makes use of three data collection techniques; Leica total station surveys, GPS, and GBInSAR. The analysis of slope deformation patterns requires that these data sources be amalgamated into a single data set; this maximizes spatial coverage of the available data across the full extent of the landslide mass. Data amalgamation has taken into account the spatial distribution and temporal period of data collection for each survey technique. All data has been checked for inconsistencies in deformation rates measured at similar locations by different

* This Chapter has been submitted for publication as:
Kalenchuk K.S., Diederichs, M.S., Hutchinson, D.J. and Barla, G. Integration of three sources of slope monitoring data to characterize landslide behaviour, Beauregard Landslide, Northwestern Italy. Submitted to Natural Hazards manuscript # NHAZ1308: 13 manuscript pages.
techniques. The final resolution of slope deformation patterns shows the highest observed rates are through the upper portion of the slope, with relatively slow rates though the central region, moderate rates along the southern portion of the landslide toe, and negligible deformation along the north toe.

7.2 Introduction

Continuous monitoring of massive slow moving landslides is a common approach to hazard management. Monitoring the behaviour of large-scale instabilities provides data that can be used in the study of slope mechanics and to assess how changing site conditions, such as ground water levels, influence slope behaviour. Many approaches to slope monitoring are available; these can range from traditional techniques, such as total station surveys and inclinometers, to state-of-the-art approaches such as interferometric synthetic aperture radar (InSAR) and light detection and ranging (LiDAR). Each of these approaches has their advantages and shortfalls; the most obvious distinction between traditional and state of the art techniques is the cost versus data accuracy and resolution comparison. Equipment acquisition and operation for more traditional approaches are often significantly less expensive than more state of the art monitoring systems which offer higher data resolution and improved accuracy.

Regardless of the acquisition technique, data is only valuable when it can be correctly interpreted. Analyses of landslide behaviour must give consideration to the spatial and temporal coverage of slope monitoring data. For instance, consider the data provided by a total station automated to complete daily surveys compared to data acquired by annual LiDAR surveys. The total station would provide excellent temporal coverage, allowing for assessment of seasonal fluctuations, while the LiDAR data would only provide
information on how the slope has deformed over an entire year. However, the LiDAR data would provide full spatial coverage of the entire slope while the total station only records movement at discrete target locations. Where multiple acquisition techniques are utilized, different types of data can be amalgamated to better resolve a complete picture of slope behaviour. This is the case at Beauregard Landslide where slope monitoring practices have incorporated both traditional and state of the art monitoring techniques including total station Leica surveys, GPS (Global Positioning System) surveys and GBInSAR (Ground-Based Interferometric Synthetic Aperture Radar). This paper reviews how data from multiple survey sources at the Beauregard Landslide have been amalgamated to analyze overall slope deformation.

7.3 Case Study

The Beauregard Landslide is a massive deep seated gravitational slope deformation (DSGSD) located in northwestern Italy in the Aosta Valley (Dora di Valgrisenche river) (Figure 7.1). An arch gravity dam is located at the toe of this extremely slow moving landslide, and continued loading of the dam structure by slope instability has created some concern for dam integrity. In response to these concerns, the Beauregard Landslide has been monitored since 1969, and the reservoir water level has been kept well below designed operation levels. In recent years, as part of an extensive landside investigation program, significant expansion of the slope monitoring practices have incorporated a number of survey techniques including total station Leica surveys, GPS surveys and GBInSAR. Each survey technique returns slope deformation rates, however the spatial extent and temporal period of slope monitoring vary between them.
Due to the differing spatial coverage of each of the data acquisition techniques used at Beauregard Landslide, it was necessary to integrate all three data types for a thorough
assessment of slope behaviour. This paper reviews this data integration, discusses challenges associated with the combination of different data sources and presents spatial deformation patterns observed at Beauregard Landslide.

7.4 Data Description

In 2003, an electronic theodolite (Leica total station) was installed to survey 19 points, 15 of which are located on the main landslide mass. Survey targets are primarily located near the landslide toe (Figure 7.1), and data is recorded on a daily basis throughout the year. This data, as illustrated using K17 in Figure 7.2, is recorded as the distance from the total station in directions parallel to slope dip, parallel to slope strike and the elevation. The average rate of displacement per year, at each target location, is resolved from the slope of linear trend-lines fit through each of the three data components.

Figure 7.2: Leica survey data collected at target location K17 with trend-lines to resolve displacement rates over the 2004-2009 time period.
The Leica data displayed in Figure 7.2 shows a long-term deformation trend with regular seasonal fluctuations. The seasonal fluctuations, observed at all Leica data points, are related to change in rockmass temperature and are not artifacts of atmospheric conditions influencing data collection. This has been confirmed by looking at the magnitude of seasonal fluctuation relative to target distance from the Leica station. If atmospheric conditions were influencing the data collection and generating artificial seasonal trends, instruments closer to the Leica station would have smaller amplitude seasonal fluctuations than those survey targets located further away. Figure 7.3 shows data points representing the average seasonal fluctuation for individual instruments illustrating that there is no correlation between distance to the Leica station and magnitude of seasonal trends at individual survey points. Each data point is an average value for individual instruments.

Figure 7.3: Poor correlation between the magnitudes of seasonal fluctuations versus the nominal distance of survey targets from the Leica total station indicates that atmospheric conditions are not contributing to the observed seasonal fluctuations.
GPS surveys were completed on August 9, 2000, August 11, 2004, November 9, 2004, June 19, 2007 and September 26, 2007. Measurements were taken at nine points on the west side of the valley and one fixed location on the opposite side of the valley (report 0109MO); seven of the nine GPS points located on the west valley slope are located on the slide mass (Figure 7.1). Due to contributions to displacement data from temperature changes in the rockmass and atmospheric conditions it is necessary to compare GPS surveys that have been completed at approximately the same time of year. Data from the two autumn periods, November 9, 2004 and September 26, 2007, are used to define the rate per year at GPS stations (Table 7.1).

Table 7.1: Movement rates defined by GPS data collected November 9, 2004 and September 26, 2007.

<table>
<thead>
<tr>
<th>GPS Survey Point</th>
<th>Movement Rate (mm/yr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>GPS1</td>
<td>5</td>
</tr>
<tr>
<td>GPS2</td>
<td>9</td>
</tr>
<tr>
<td>GPS4</td>
<td>6</td>
</tr>
<tr>
<td>GPS5</td>
<td>7</td>
</tr>
<tr>
<td>GPS6</td>
<td>7</td>
</tr>
<tr>
<td>GPS7</td>
<td>3</td>
</tr>
<tr>
<td>GPS8</td>
<td>2</td>
</tr>
</tbody>
</table>

GBInSAR monitoring was carried out in 2008 between June 18 and October 17, collecting data on 20 minute intervals. The GBInSAR system was installed on the right abutment of the dam at 1775 m a.s.l (Figure 7.1), and provides coverage of all areas of the slope within line of sight that have exposed outcrop (Figure 7.4). Data is most dense at higher elevations where rock exposure is not hindered by vegetation, while at lower elevations GBInSAR data is limited to isolated boulders visible through forest cover (Barla et al. 2009).
GBInSAR data has been collected over a specific portion of one year (June to October, 2008) making it difficult to define annual deformation rates. The four month total displacements cannot be directly computed to rate per year because slope deformations are not constant year round. It is known that most slide deformation occurs during summer months in response to seasonal change in ground water levels (Miller et al. 2008); and seasonal fluctuations in response to rockmass temperature have been
identified in the Leica data. Therefore, to define annual deformation rates based on total displacements measured over the four month period it is necessary to define the difference in rate between the June to October period and the long-term average. To do this Leica data has been studied closely. Figure 7.5 demonstrates the difference in rate between June and October and the long-term trend. The steeper slopes of short-term trend-lines are indicative of higher rates of displacement. By analyzing all Leica data in this fashion is has been concluded that rates measured between June and October are, on average, 36% higher than long term trends, with a standard deviation of 34%. It is assumed that this is a global discrepancy which would be observed across the full landslide extent if data availability permitted. Based on this assessment of the Leica data it is a reasonable assumption that GBInSAR data measured over the 4 month summer/fall period also over-estimates the annual displacement rates by 36% and the raw data is therefore adjusted accordingly to assume annual deformation rates.

Figure 7.5: Leica survey data collected at target location K17 demonstrating that June-October deformation rates (absolute slope of the trend-lines) are greater than long-term trends.
7.5 Data Integration

In the analysis of massive landslide slope deformation, the spatial coverage of slope monitoring data should be maximized. This is often difficult due to limitations with specific slope monitoring techniques. For instance, Leica surveys and GPS coverage are limited by accessibility to specific regions of the slope, and GBInSAR is limited to regions with rock exposure within the station line of sight. At Beauregard, each data type is concentrated through specific areas of the slope: Leica near the toe, GPS through the central portion of the slide and GBInSAR at higher elevations. To achieve full spatial coverage of the entire landslide, all data sources have been integrated into a single data set. Integrated data has then been contoured to define a slope deformation pattern across the full extent of the landslide mass.

The first step in data integration is to determine if any inconsistencies occur in deformation rates measured at similar locations by different techniques. Such inconsistencies have been identified at two locations:

- K19, located in the upper portion of the slope, records displacements at a rate of 6.8 mm/yr, while GBInSAR data in close proximity record rates of 8-9 mm/year. Based on findings by Barla et al. (2009) data recorded at Leica survey points more than 1.5 km from the total station are less reliable than interferometer measurements. Therefore, it is likely that GBInSAR in this area of the slope provides better data than K19 Leica point.

- GPS7, near the south landslide boundary, records displacement rates of 3 mm/year and nearby Leica targets K11 and K17 record rates of 7.2 and 8.5 mm/year. The low rates observed at GPS7 may be attributed to the close proximity of this point.
Based on these explanations, data from K19 and GPS7 have been removed from the integrated data set.

Another data feature to consider when integrating different monitoring sources is data density. GBInSAR data provides full spatial coverage in those areas of exposed rock within the instrument field of view, while GPS and Leica surveys return point data. In order to integrate all three sources to a common data set, GBInSAR data has been resolved from raster format to gridded data points. In doing this, consideration has been given to the appropriate grid density (10 m, 50 m, 100 m, 150 m and 200 m) to which GBInSAR data should be converted. Cumulative distribution curves (Figure 7.6) demonstrate the percentage of gridded data points which measure displacement at or below specific values. This technique for illustrating data demonstrates that as less samples are taken per area of the raw raster data (increased grid spacing) detail in the returned data set decreases. For example the 150 m and 200 m grid spacing data jump very abruptly from 0% less than to 100% less than, while data spaced at 10 m, 50 m, and 100 m return smoother distributions of total displacements. The similarity in % less than distributions of the 10 m, 50 m and 100 m spaced data indicates that 100 m spacing is sufficiently dense to represent the raster data without loss of detail which would otherwise over-smooth the final gridded data-set. Further review of the gridded data statistics (inset to Figure 7.6) indicate that with decreased grid densities, the extreme
minimum and maximum values in observed deformation rates are lost, while the average is essentially constant and standard deviation varies according to the data range.

The selection of an appropriate grid density is important, because when contouring is completed, areas with less dense data will appear, relatively, over-smoothed. To illustrate this phenomenon, the 10 m, 50 m and 100 m grid spacing interferometer data sets have been independently integrated with GPS and Leica data (Figure 7.7), and deformation rates have been contoured using a multiquadratic radial basis function (Aguilar et al. 2005, Hardy 1990). This function, available in SURFER software version 8.05 (Golden Software, Inc. Golden, Co., USA, 2004), has been selected based on minimum error returned from cross-validation. The 10 m gridded GBInSAR data returns very detailed contours through the upper portion of the landslide while areas near the toe appear smoother. This gives an apparent impression of complex behaviour with localized variations in deformation rates through the upper slope while the lower regions of the
landslide mass appear to be controlled by more global deformation patterns. This discrepancy in behaviour between the upper and lower slope is a function of data density and does not necessarily reflect the true spatially discrimination of deformation patterns. The 100 m grid GBInSAR data returns a smoothed contour plot across the entire landslide, removing this apparent discrepancy. This smoothing of slope deformation through the upper portion of the slope is reasonable because the focus of this landslide study is on interpreting overall slope behaviour. If the study focused on a more detailed analysis of slope behaviour through specific landslide regions, it may be more suitable to use the denser data where available.

Figure 7.7: (top) Distribution of data across the Beauregard Landslide mass, and (bottom) contoured displacement rates for data sets with varying GBInSAR data density (a) 10 m, (b) 50 m and (c) 100 m.
For comparison purposes the slope deformation patterns have also been interpreted using sub-sets of the displacement data: (a) Leica only, (b) GPS only, (c) GBInSAR only, (d) Leica and GPS, (e) Leica and GBInSAR, (f) GPS and GBInSAR. Figure 7.8 illustrates these interpretations and it is obvious that sparsely sample regions are over-smoothed, and as such it is deemed necessary to utilize all of the available data sets.

Figure 7.8: Comparison of slope deformation patterns interpreted using sub-sets of the Beauregard displacement data: (a) Leica only, (b) GPS only, (c) GBInSAR only, (d) Leica and GPS, (e) Leica and GBInSAR, (f) GPS and GBInSAR.

To further the analysis of overall slide behaviour, deformation rates are standardized to describe each specific data point in terms of plus or minus standard deviation from the data set average (Figure 7.9). Positive and negative standard deviations indicate deformation rates above and below the global average, respectively. This provides
insight into spatial variation in rate of movement. As demonstrated by Kalenchuk et al. (2010a – Chapter 3), this approach is particularly important for the comparison of field data to numerical simulations, where absolute deformation rates cannot be directly compared because field data is in mm/year and numerical data is in displacements/time step. Standardization of the Beauregard displacement rates is sensitive to the data density. For instance with 10 m and 50 m grid spacing, dense GBInSAR data throughout the upper portion of the slope is heavily weighted relative to sparser data regions. As a result, the displacement rate standard deviation from the mean data is skewed by those rates observed at these higher elevations.

Figure 7.9: Representation of overall slide behaviour with displacement rate standard deviation from the mean data sets with varying GBInSAR data density (a) 10 m, (b) 50 m and (c) 100 m.

Based on this analysis, the overall Beauregard Landslide behaviour can be best analyzed by amalgamating Leica and GPS data with GBInSAR data gridded at 100m spacing. The highest observed rates of deformation occur in the upper region of the landslide, slower movements occur through the central slope, moderate deformation rates are observed near
the southern toe and the north toe is characterized by extremely slow deformation rates. These findings are applied to further research in how the kinematics of Beauregard Landslide are influenced by geological, geomechanical, and ground water conditions by Kalenchuk et al. (2010b – Chapter 8)

7.6 Conclusions

Slope monitoring is a key component in hazard management for massive slow moving landslides. Data pertaining to deformation rates, and how these rates change over time, is required in order to gain understanding of slope deformation processes and landslide mechanics. Deformation at Beauregard Landslide has been monitored by multiple data acquisition techniques including Leica surveys, GPS surveys and GBInSAR. Each acquisition method provides deformation rate data over specific regions of the slope. In order to achieve full spatial coverage of the entire landslide it is necessary to amalgamate these varying data sources.

The data amalgamation process presented here accounts for discrepancies between data types. Particular data sources are more accurate in different regions of the slope depending on, for instance the distance to survey monuments for Leica surveys. Poor-quality data has been recognized and removed from the overall data set prior to interpretation of deformation patterns.

Interpretation of data is optimized when the density of data is consistent across the entire slope. When the data distribution is clustered, dense data regions highlight localized complexities in global landslide behaviour, while areas of low-data density appear over-smoothed. Localized complexities are real; however, they are not representative of the
overall slope deformation. To achieve a global view of slope displacement patterns at Beauregard, GBInSAR data density has been reduced by converting raster to point format data with similar spacing to the GPS and Leica data. While the densest possible data has not been required for the analysis presented in this paper, that data should be utilized to investigate localized slope activity.

Slope deformation patterns have been presented in terms of displacement rates, as well as displacement rate standard deviation from the mean. Both approaches for data presentation are utilized for interpreting spatially discriminated landslide behaviour. The latter, standardized approach is particularly important for comparing field data to numerical simulations where deformation rates are not directly correlated. It has been concluded that the Beauregard Landslide shows high deformation rates through the upper portion of the slope and moderate rates near the southern toe. The central portion of the slope is slightly slower and movements observed at the north toe are effectively negligible.

7.7 Acknowledgements

The authors would like to thank Giovanni Barla, Marco Barla and Giovanna Piovano for data access. This work has been made possible through contributions by NSERC and GEOIDE.

7.8 References


Kalenchuk, K.S., Diederichs, M.S. and Hutchinson, D.J., 2010a. Three-dimensional numerical simulations of the Downie Slide to test the influence of shear surface geometry and heterogeneous shear zone stiffness. Submitted to Computational Geosciences in manuscript # COM368: 24 manuscript pages.


CHAPTER 8*

A Geomechanical Analysis of the Beauregard Landslide Using Three-Dimensional Numerical Models

8.1 Abstract

Three-dimensional numerical models have been developed to reproduce observed slope deformation at the Beauregard Landslide. In doing so, model calibration has accounted for three-dimensional interpretations of shear surface geometry, spatial variation in shear zone frictional strength, and the mechanical interaction between different regions of the landslide mass. Trained models are used to test the slope response to hypothesized water table changes resulting from reservoir filling and the development of a slope drainage system. Modelling results indicate that with slope drainage, some increase in reservoir level may be achieved without exceeding deformation rates simulated under current water table conditions. However, simulation of the reservoir at full capacity, even with a drainage system in place, accelerates slope displacement rates beyond those produced in simulations with current water table conditions.

* This Chapter has been submitted for publication as:
8.2 Introduction

The Beauregard Landslide is a DSGSD (Deep-Seated Gravitational Slope Deformation) located in the Aosta Valley on the west bank of the Valgrisenche River (Figure 8.1). Continuous creep deformation was first recognized during filling of the Beauregard Reservoir. This massive rockslide impinges on the left abutment of a gravitational arch dam and continued loading of the dam has caused some closure of the arch, posing a hazard to the dam safety and compromising reservoir operations.

Figure 8.1: (top) Beauregard landslide as viewed from the east side of the valley, with inset map showing location in northwestern Italy. (bottom) The Beauregard Dam as viewed from upstream; reservoir levels are restricted to well below design capacity due to loading by the landslide which impinges the left abutment.
Complex slope mechanics characterize landslides of this scale, for example: Downie Slide (Kalenchuk et al. 2010a – Chapter 4), Vaiont (Hendron and Patton 1987, Semenza and Ghirotti 2000) and Frank Slide (Benko and Stead 1998) to name a few. In many cases, different regions of a large slope instability exhibit variable rate and direction of deformation, as well as composite failure mechanisms. Knowledge of massive landslide geomechanics, and an understanding of how different slope regions behave and interact, improves the effectiveness of hazard management and the design and application of engineered mitigation approaches.

The research presented in this paper focuses on the continuous very slow to extremely slow displacements at Beauregard; assessing which geomechanical factors control spatial variation in deformation magnitude, and how ground water boundary conditions control change in rate. A mixed continuum-discontinuum numerical modelling approach is utilized to simulate the dynamics of a deformable rockmass interacting with in situ material along a discrete failure surface. Three-dimensional simulations are required in order to account for the complexities in geometry and spatial variability in material strength parameters. In addition to studying how the three-dimensional geometry of the basal slip surface and the distribution of shear zone strength parameters influence slope mechanics, these numerical simulations give consideration to the mechanical interaction between landslide zones.

This work aims to improve our understanding of the Beauregard Landslide slope mechanics and also to demonstrate the necessity for developing trained, sophisticated three-dimensional numerical simulations capable of reproducing observed slope behaviour. Trained models can be utilized for predictive purposes; for example to assess
the effectiveness of slope drainage. Given the physical scale, complexity and extent of development of this landslide, the complete arrest of the instability is not likely possible, however the development of drainage infrastructure may be able to slow deformation rates. Forward modelling does not aim to define absolute values of changed displacement rates; rather, the goal is to assess which areas of the slope are most influenced, and to assess if drawdown would be expected to have a significant influence on slope deformation.

8.3 Background

Site investigation for dam construction was initiated in 1950, involving boreholes and two tunnels near the dam at 1772 and 1680 m a.s.l. Investigation brought to light the fact that the left valley wall has considerably poorer rockmass quality than the right. The right side of the valley is massive and of good quality, reported as “sound, compact, resistant and impervious” (SIP Group 1952). The left valley wall “shows frequent and irregular inclusions of mylonitic material due to local fine crushing of the crystalline schists” (SIP Group 1952). In addition to the mylonitic features, there are various fault systems on the left side of the valley. Further compromising the left abutment rockmass quality is a massive rockslide of mica schist which overlays a deep pocket of glaciofluvial sediments. The glaciofluvial material reaches a maximum depth of 150 m into the slope and extends to a distance of about 80 to 90 m from the abutment (Barla et al. 2006). In order to improve rockmass quality, giving solid support to the left dam abutment and reducing permeability, the glaciofluvial sediments were completely replaced by concrete to a depth of 200 m into the slope in addition to consolidation grouting in the left abutment and construction of a grout curtain around the dam to 100 m below the river bed (Barla et al.
Construction of the hydroelectric infrastructure and the dam were completed by the late 1950s. Reservoir filling began in 1959 and was completed by 1968.

The landslide was first recognized during reservoir filling, when accelerated movements were observed in response to toe inundation. In 1969, the Italian Dam Authority enforced restrictions on the reservoir capacity. Initially designed to operate at 1770 m a.s.l., (70 million cubic meter capacity), the reservoir level was restricted to 1710 m a.s.l. (6.8 million cubic meters) with an absolute maximum of 1715 m a.s.l. during flood events. Further restrictions were applied in the 1998, limiting the reservoir to 1705 m a.s.l. with an absolute maximum of 1710 m a.s.l. (Barla et al. 2006). The reservoir has operated between 1700 and 1705 m a.s.l. since 1997, with a temporary drawdown in 2005 for maintenance purposes.

The Beauregard Dam was operated by the Italian Electric Energy Company (ENEL) until July, 2001 when it was acquired by Compagnia Valdostana delle Acque (CVA). Over the operating life of the hydroelectric project, slope deformation has loaded the left abutment and the dam arch has progressively closed. In response to safety and operational concerns, CVA initiated a site investigation in 2002. The investigation aimed to improve understanding of the DSGSD and the interaction of the slope instability with the dam. This has comprised geological, geomorphological, hydrological and geotechnical studies, involving in situ and laboratory testing and geophysical seismic investigations. The Beauregard Landslide has been monitored since 1969 and as part of the CVA investigation program, recent expansion of the slope monitoring practices have incorporated a number of survey techniques including total station Leica surveys, GPS (Global Positioning System) surveys and GBInSAR (Ground-Based Interferometric
Synthetic Aperture Radar). Further discussion of this data is provided in Barla et al. (2009) and Kalenchuk et al. (2010b – Chapter 7).

The CVA site investigation also initiated slope stability analyses for Beauregard. Barla et al. (2006) utilized FLAC to reproduce mechanisms driving instability and simulate interaction between the upper and lower portion of the slope. Miller et al. (2008) completed two-dimensional limit equilibrium analyses for the lower portion of the slide; testing linear and non-linear shear strength models and critical slip surface geometries. Using a non-linear power model for shear strength to predict the extent of failure resulted in a two-dimensional shear zone geometry that is in good agreement with subsurface data obtained from boreholes and geophysical surveys. Stochastic slope stability analyses were used to explore time-based probability of sliding for seasonal high (summer time) ground water levels, and reduced displacements were predicted for a proposed ground water drainage adit system (Miller et al. 2008).

8.4 Geological, Structural and Morphological Setting

The landslide is composed of fractured gneiss and mica-schist of the Gran San Bernardo Series. Instability is facilitated by a well developed schist foliation which dips 23° to 28° towards the valley (Miller et al. 2008). Two boreholes (S1/03 and S1/04) were drilled in the lower portion of the landslide as part of the CVA site investigation program. These crossed the basal slip surface, which is described as a zone of sheared and crushed rock, locally reduced to soil-like material with silt and clay (Barla et al. 2006). Joints and shears are predominantly steeply dipping and strike valley parallel, and minor jointing and shears are roughly orthogonal to these prominent features (Barla et al. 2006).
The Beauregard Landslide measures approximately 1700 m north to south across the toe and 2400 m from toe to head scarp. It extends from 1700 to 3200 m a.s.l., with an estimated maximum thickness of 260 m (Miller et al. 2008) and an approximate volume of 400 million m$^3$. The west and north landslide boundaries are defined by a major scarp, while the south slide boundary is defined by less obvious trough features. Landslide morphology features internal scarps, ridges and troughs, open tension cracks and trenches. Based on morphological features, as well as seismic and borehole data, a number of zones within the landslide have been identified (Figure 8.2). The upper extents of the landslide features a highly fractured and weathered rockmass; a large mass referred to as the Scavarda Ridge has broken away from the head scarp (Barla et al. 2009). Below this ridge, localized toppling failure and rockfalls have contributed to talus accumulations. The Bois de Goulaz Ridge and the Bochat Ridge are found in the central area of the slide. Tension features are prominent upslope from both. The lower region of the landslide, bound to the west by a well developed scarp near Alpettaz, features numerous minor scarps and counter-scarps striking valley parallel (Barla et al. 2009).
Figure 8.2: Geological map of the Beauregard Landslide (geological mapping has been provided by Barla 2009)
8.5 Ground water

Sixteen piezometers have been monitoring ground water levels at Beauregard since 2003. For the purpose of geomechanical numerical modelling, the piezometric data has been used to interpret ground water levels across the full extent of the landslide. It is important to note that this interpretation is not meant to be a detailed hydrogeological analysis, but rather it aims to generate a reasonable state that can be used in numerical simulations. It is probable that the hydrogeology is much more complex, as ground water levels may be locally influenced by faulting and shears, and multiple water tables may exist due to confinement by low permeability regions, such as clay rich shears or landslide areas under compression.

Piezometers are clustered near the landslide toe, with no instruments located at higher elevations due to accessibility limitations (Figure 8.3); as such water levels through the upper portion of the slope are largely inferred. The majority of the piezometers are located within the landslide mass, above the lower shear zone. Given the limited data for water levels below the basal slip surface, it is necessary to assume that pore pressures acting on the basal slip surface are related to water table levels within the slide. This is considered a reasonable assumption given that data from two boreholes, where piezometers are located both above and below the basal slip surface (for example PZ7N and PZ6N as shown in cross section, Figure 8.3), show similar piezometric heads across the shear zone.

Figure 8.3 illustrates the location of piezometers in cross section. It is apparent that some piezometers are sampling perched water tables where significant difference in measured water table levels are observed between boreholes at close proximity (for example PZ1
vs. PZ6N and PZ7N). It is assumed that perched water tables are not representative of the overall ground water conditions, and as such, data where a perched water table is suspected are not used for water table interpretation. A sharp curve in the water table profile is also apparent in section, this is interpreted to be related to localized drawdown near the tunnel at 1772 m a.s.l..

Figure 8.3: Cross-section illustrating the location of piezometers, an inferred water table measured by a number of instruments (M17bis, PZ6, PZ7, CL3 and S1/04) and a perched water table sampled by one piezometer (PZ1). Inset shows plan view of piezometer locations and the position of the cross-section plane within the landslide boundary.

The water table utilized in numerical models has been interpreted from the maximum seasonal level recorded at each piezometer. Analysis by Barla et al. (2006) correlated increased movements during summer months with ground water rise associated with increased infiltration into the slide mass. The minimum level of ground water conditions is unknown because 11 of the instruments report minimum levels at the installed elevation of the piezometer tip. These instruments are not deep enough to measure true minimum ground water levels.
SURFER software version 8.05 (Golden Software, Inc. Golden, Co., USA 2004) has been used to generate a three-dimensional water table across the full extent of Beauregard Landslide. The water table is assumed to be a continuous. A minimum curvature gridding algorithm (Smith and Wessel 1990) has been applied to the available data; this algorithm was selected based on minimized cross-validation error. The resulting water table is illustrated in Figure 8.4; note that artesian conditions are interpreted through the central portion of the slope. The lack of data through this area and the upper portion of the slope make ground water interpretations difficult. These artesian conditions are considered reasonable by observation of streams on the landslide; however the exact locations of seeps have not been formally mapped.

Figure 8.4: Water table interpretation using minimum curvature algorithm based on piezometric data.
8.6 Landslide Monitoring and Slope Behaviour

Continuous monitoring of massive slow moving landslides is a common approach to hazard management. Monitoring the behaviour of large-scale instabilities provides data that can be used in the study of slope mechanics and to assess how changing site conditions, such as ground water levels, influence slope behaviour. Slope monitoring practices at Beauregard Landslide have incorporated both traditional and state of the art monitoring techniques including total station Leica surveys, GPS surveys and GBInSAR (Figure 8.5).

![Figure 8.5: Displacement rates measured by Leica, GPS and GBInSAR in mm/year (modified after Kalenchuk et al. 2010b – Chapter 7).](image)

The analysis of massive landslide slope deformation is optimized by spatially maximizing coverage of slope monitoring data. At Beauregard, each data type is concentrated
through specific areas of the slope; Leica near the toe, GPS through the central portion of
the slide and GBInSAR at higher elevations. Therefore, to achieve full spatial coverage
of the entire landslide, Kalenchuk et al. (2010b – Chapter 7) have amalgamated these
survey sources and analyzed overall landslide deformation. Integrated data has been
contoured to define a slope deformation pattern across the full extent of the landslide.
Deformation patterns are presented in Figure 8.6 as contoured mm/yr and as displacement
rate standard deviations from the mean, where local displacements are described in terms
of plus or minus standard deviation from the data set average. This standardized data
provides insight to spatial variation in terms of rate of movement. This approach, as
demonstrated by Kalenchuk et al. (2010c – Chapter 3), is particularly important for the
comparison of field data to numerical simulations where absolute deformation rates
cannot be directly compared, because field data are in displacements per year and
numerical data are in displacements per time step. The highest observed rates (classified
as extremely slow according to the Cruden and Varnes (1996) classification system) of
deformation at Beauregard Landslide occur in the upper region of the slope, slower
movements occur through the central slope, medium deformation rates are observed near
the southern toe and the north toe is characterized by negligible displacements.
8.6.1 Temporal Variations in Slope Behaviour

An assessment of monitoring data carried out by Miller et al. (2008) has correlated variable slope deformation rates with changing ground water levels. As an example, Figure 8.7 illustrates Miller et al.’s (2008) comparison of displacements measured on a borehole pendulum (measured left to right, perpendicular to the valley axis) to depth of snow cover. Accelerated deformation rates were observed during reservoir filling (1959-1968); this lead to the water level restrictions imposed in 1969 (Barla et al. 2006).
Seasonal slope accelerations coincide with summer months (May-June to September-October) where water levels are elevated by snowmelt and the subsequent infiltration of runoff. Negligible displacements are observed from late autumn to late winter. Wet years (1975, 1977-78 and 1994-95) also show accelerated deformation rates relative to drier years (1989-93). Knowledge that slope behaviour at Beauregard is influenced by ground water levels has encouraged the proposal for slope drainage as a means to slow deformation rates.

Figure 8.7: Beauregard slope movements as measured at various elevations on a borehole plumb-line show correlation with ground water levels. Seasonal slope accelerations coincide with elevated water tables during summer months. Inset shows location of borehole plumb-line (modified after Miller et al. 2008).

### 8.7 Numerical Modelling

Three-dimensional numerical models have been developed and calibrated to reproduce observed slope behaviour for the Beauregard Landslide. The mixed continuum-
discontinuum code 3DEC (3-Dimensional Distinct Element Code) (Itasca Consulting Group, Inc. Minneapolis, Minnesota 2003) is used to facilitate large strains along shear zone interfaces while allowing the landslide body to incur plastic deformation. Boundary conditions for these models include: gravity field stresses, a buttress load where the reservoir has inundated the toe of the landslide, and pore pressures for specified ground water levels. For a more detailed explanation of procedures for the development of sophisticated three-dimensional landslide models in 3DEC readers are referred to work by Kalenchuk et al. (2010c – Chapter 3). Calibration of the Beauregard Landslide models accounts for shear zone geometry, spatial variation in shear zone strength parameters and the interaction between landslide zones. Trained models capable of reproducing observed, steady-state deformation patterns are used to test the effectiveness of drainage system development as an engineered approach to hazard mitigation.

8.7.1 Landslide Geometry

Three-dimensional geometries of the basal slip surface have been interpreted from three geophysical sections and the location of two borehole intercepts (Figure 8.8) following the procedure proposed by Kalenchuk et al. (2009 – Chapter 2) for geologically realistic interpretations of shear zone geometries. Figure 8.9 illustrates geometries returned by two interpolation algorithms, a minimum curvature (Smith and Wessel 1990) and a multiquadratic radial basis function (Lazzaro and Montefusco 2002, Aguilar et al. 2005). These algorithms have been selected for application to numerical simulations based on minimum cross-validation error and visual assessment of spatial patterns. The simulation and comparison of both geometries tests the sensitivity of slope behaviour to variations in the interpreted shear zone geometry.
Figure 8.8: Shear zone location has been identified in seismic sections and borehole intercepts (courtesy of Barla et al. personal communication 2008). (left) Location of seismic lines and boreholes and (right) sample seismic data illustrating location of borehole intercept, basal shear surface, and major faults.

Figure 8.9: Basal slip surface geometries interpreted using (a) a minimum curvature algorithm and (b) a multiquadratic radial basis function.
Slope deformation is sampled in the numerical models at the same geographic locations as where field monitoring data is available. This ensures consistency in data density and distribution. Slope deformation, measured in the field as displacements per real time, and in models as displacements per time step, are represented as displacement rate standard deviations from the mean at each data point. Model output is compared to field data quantitatively using $R^2$ values describing the correlation between modelled and measured displacement rate standard deviations from the mean. Qualitatively, comparison is done visually by assessing the spatial patterns of contoured rate standard deviations. Figure 8.10 illustrates the quantitative and qualitative comparison of models with varying basal shear surface geometries. The minimum curvature geometry returns the highest $R^2$ value and, by visual comparison of surficial deformation patterns returned for each model to field data (Figure 8.6), produces the best representation of observed slope deformation patterns and is therefore taken as the truest three-dimensional interpretation of shear surface geometry.
Figure 8.10: (top) Displacement rate standard deviation from the mean measured in the field vs. modelled returns $R^2$ values used to quantitatively compare various model simulations, $R^2$ values closer to 1 indicate better correlation. (bottom) Contoured displacement rate standard deviations from the global mean for used for visual comparison between modelled and measured data.
8.7.2 Constitutive Models and Material Properties

The landslide mass and in situ material below the landslide are defined as continuum materials and the Mohr-Coulomb failure criterion is applied to them. Material properties are summarized in Figure 8.11; these have been assigned to represent reasonable values based on the geological model produced by Barla et al. (2006), results of laboratory testing (laboratory testing results were provided by Barla 2009) (Figure 8.12 and Table 8.1) and model sensitivity testing. In situ material is described by Barla et al. (2006) as a good quality, undisturbed and blocky rockmass and the landslide material is a poor quality, weak and disturbed rockmass containing crushed rock and soil-like material. $E_{rm}$ is defined for the continuum materials using the Generalized Hoek-Diederichs equation (Hoek and Diederichs 2006):

$$E_{rm} = E_i \left( 0.02 + \frac{1 - D^2}{1 + e^{(15D - GSI)/11}} \right) \quad (8.1)$$

Where the disturbance factor, $D$, is assigned a value of zero as the fractured nature of the rockmasses are accounted for in the GSI (Geological Strength Index (Hoek et al. 1995)) values.

Equivalent Mohr-Coulomb parameters (friction angle and cohesion for the landslide mass and friction angle for the in situ material) and tensile strength for the landslide mass have been computed by RocLab (RocScience, Inc. 2007). Cohesion and tensile strength values for in situ material are not taken from the geological model; these are assigned values of 1 GPa and 100 MPa respectively. These unrealistically high values ensure that no failure occurs outside of the landslide boundaries; a reasonable condition given that the scope of
this modelling is to study the behaviour of an existing landslide rather than the propagation of new instability. Both the in situ and disturbed materials are assigned zero dilatancy.

Figure 8.11: (top) Schematic illustrating material properties utilized in 3DEC simulations, (middle) a typical cross section through the landslide model and (bottom) isotropic view of the 3D Beauregard model looking southwest (modified after Kalenchuk et al. 2010d).
The landslide mass and the in situ material interact along discontinuum joint elements which define the landslide shear surfaces. These are controlled by the Coulomb-slip constitutive model (Itasca Consulting Group, Inc. 2003). Testing has been done on undisturbed samples of shear zone materials, the schist foliation and along joints; the results are summarized in Table 8.1. In addition to triaxial testing, direct shear and creep direct-shear tests have been completed for specific effective stress states using a customized direct-shear apparatus (Miller et al. 2008). The upper portion of the slope is hypothesized to behave in a more brittle manner than the lower landslide regions. To
define reasonable base case input parameters for numerical simulations, the shear zone material strength through the upper portion of the slope is assumed to be similar to that of the schistose material \( (\varphi_p = 25^\circ, \varphi_r = 19^\circ, c_p = 140 \text{ kPa}, c_r = 130 \text{ kPa}) \). The shear zone is interpreted to be more developed throughout the lower portion of the slope and here strength parameters are assumed to be similar to samples of shear zone material tested by triaxial testing \( (\varphi_r = 27^\circ, c_r = 133 \text{ kPa}) \).

As mentioned, considerable work was completed as part of dam construction to improve the rockmass quality near the left abutment, including the complete replacement of a large pocket of glaciofluvial material overrode by the landslide. The concrete mass extends to depths of 200 m into the slope in the vicinity of the left abutment. To account for this large concrete mass in the numerical models it is assumed that the shear surface has been completely replaced through this portion of the slope and stronger material properties \( (\text{friction} = 47^\circ, \text{cohesion} = 1 \text{ MPa}, \text{tensile strength} = 1 \text{ MPa}) \) are therefore applied to the basal slip surface in close proximity to the dam.

Sensitivity testing has been completed to better refine shear zone strength parameters. A suite of numerical models, using the minimum curvature basal slip surface geometry, have tested how simulated slope behaviour changes when the models are assigned a range of residual frictional values \( (25^\circ \text{ to } 29^\circ \text{ for the lower portion of the slope and } 17^\circ \text{ to } 21^\circ \text{ for the upper portion}) \). Cohesion has been kept constant at 130 kPa (residual) for the lower slope and 140/130 kPa (peak/residual) for the upper slope. Figure 8.13 illustrates the modelling results; increases in frictional strength through the upper and lower portions of the slope return localized deceleration of displacement rates. It has been concluded, based on \( R^2 \) values and visual assessment of contoured displacement rate standard
deviations from the mean, that a residual friction of 25° be applied to the lower slope with peak and residual friction equal to 25° and 19°, respectively, applied to the upper slope to optimize the simulation of field observations. This distribution of frictional strength is taken to represent the truest spatial distribution of shear surface character.

Figure 8.13: Contour plots of displacement rate standard deviations from the mean in numerical models with varying residual frictional strength in the upper and lower regions of Beauregard Landslide (Kalenchuk et al. 2010e).
8.7.3 Interaction between Landslide Zones

The final stage in calibration of the Beauregard models has tested how the discrete definition of multiple internal landslide zones influences slope behaviour in comparison to treating the entire instability as a monolithic mass. Figure 8.14 illustrates the base case landslide model with numerous zones as well as a simplified monolithic model. The zoned model produces better $R^2$ values and spatial deformation patterns than the monolithic equivalent when compared to field displacement data. Differential movements at Beauregard Landslide are best simulated by allowing landslide zones to displace independent of one another.

Figure 8.14: (top) Plan view of numerical models with (left) discrete landslide zones and (right) a single monolithic mass. (bottom) Contour plots of displacement rate standard deviations from the mean measured in numerical models (Kalenchuk et al. 2010e).
8.7.4 Drainage Testing

Controlling ground water levels is a common approach to managing slope deformation in massive, complex landslides. A ground water drainage system has been proposed for Beauregard in order to lower, and maintain, water table levels (Miller et al. 2008). To investigate the potential effectiveness of this engineered mitigation technique, a simplistic drainage scenario has been hypothesized by extending the 1772 m a.s.l. site investigation tunnel into the slope past the basal slip surface, and south parallel to the valley (Figure 8.15). It is assumed that this tunnel and a network of drain holes fanning from the tunnel would lower the water table locally to the tunnel elevation. This assumed change to water table levels is used to numerically test how slope behaviour is affected and a number of scenarios have been simulated including: (a) drainage with current reservoir levels (1700 m a.s.l.), (b) drainage with reservoir filled to 1735 m a.s.l., (c) drainage with reservoir filled to full capacity (1770 m a.s.l.) and (d) a full reservoir with no drainage.

Following a numerical set-up sequence designed to bring models to initial equilibrium (Kalenchuk et al. 2010c – Chapter 3), models have been run a pre-defined number of time steps (20,000) with current ground water conditions and then water table changes are applied and time stepping is continued for an additional 20,000 steps. The rate of movement is measured over the second 10,000 steps of the latter 20,000 steps, in order to assure that steady state deformation rates have been achieved. In numerical models, water table changes are instantaneous, where as in reality fluctuations would occur over some period as a function of rockmass permeability. While dynamic water levels are not simulated, it should be noted that in reality, differential changes to slope rates would
occur across the landslide mass as the ground water conditions respond more gradually to change in boundary conditions.

Figure 8.15: Hypothesized changes to ground water levels with drainage development and varying reservoir levels; (a) drainage with current reservoir levels (1700 m a.s.l.), (b) drainage with reservoir filled to 1735 m a.s.l., (c) drainage with reservoir filled to full capacity (1770 m a.s.l.) and (d) a full reservoir with no drainage.

Figure 8.16 illustrates the modelled deformation rates in mm/time step for simulation of the current water table conditions and each of the tested scenarios. Percent change in rate is also displayed; any change in rate less than 10% is interpreted to be insignificant and
representative of constant displacements. Displacement rates are sampled in the models at the location of real slope monitoring data points, this consistency would be very important for model verification if drainage system development and adjustment to reservoir levels are eventually undertaken. From these results it is apparent that the development of a slope drainage system with current reservoir levels (scenario a) would achieve slowed deformation rates across the lower and middle regions of the landslide. The most significant changes are observed across the landslide toe, and no change occurs at higher elevations where the assumed water table remains constant between pre- and post-drain periods.

When impounding a reservoir on any slope instability, it should be mandated that the reservoir level may be increased, provided stability is not compromised, and where accelerations are anticipated, engineered mitigation should aim to more than offset the anticipated slope response to toe inundation. Modelling results suggest that with slope dewatering, some increase to the Beauregard Reservoir level can be achieved safely. A simulated reservoir level of 1735 m a.s.l. (scenario b) shows minor accelerations throughout the lower portion of the slope. These changes to displacement rates are interpreted as minor, and the overall slope behaviour is largely comparable to models with current ground water conditions. Further rise in reservoir level (scenario c) to full capacity (1770 m a.s.l.) accelerates slope rates to unacceptable levels as they exceed those modelled with current water level conditions.
Figure 8.16: Modelling results for changes to ground water states. (Top left) Displacement rates in mm/time step modelled under current ground water conditions. (center) Displacement rates in mm/time step modelled for varying ground water scenarios (a) drainage with current reservoir conditions, (b) drainage with reservoir at 1735 m (c) drainage with full reservoir and (d) full reservoir with no drainage system in place.
During initial filling of the Beauregard Reservoir, slope accelerations raised concern for dam safety leading to the imposition of reservoir level restrictions. The simulation of a full reservoir (scenario d) causes considerable slope accelerations through the lower portion of the slope. Unfortunately there is insufficient monitoring data available to correlate observed and modelled slope response to toe inundation.

**8.8 Summary**

Reservoir operations and the safety of the Beauregard Dam are compromised by a massive DSGSD on the left valley wall. The CVA initiated an extensive site investigation program in 2002 involving geological, geomorphological, hydrological and geotechnical studies, in situ and laboratory testing and geophysical seismic investigations. As part of this ongoing work to improve knowledge of the Beauregard Landslide geomechanics the research presented in this paper has focused on the interpretation of landslide behaviour and the development of calibrated numerical models capable of reproducing observed slope deformation.

Continuous slope monitoring by total station Leica surveys, GPS surveys and GBInSAR has provided sufficient data to interpret modern deformation patterns across the full extent of the instability. These deformation patterns are spatially discriminated as different areas of the slope exhibit variable rates of movement. The upper portion of the slope, classified as very slow to extremely slow, is the fastest moving area of the landslide, lower rates are observed along the south toe, even lower rates occur through the central region and the north toe shows negligible deformation.
This research has investigated how continuous very to extremely slow displacements at Beauregard Landslide are influenced by geomechanical factors including shear surface geometry, spatial variation in material strength parameters and the interaction between landslide zones. Mixed continuum-discontinuum models have been developed and calibrated to simulate real deformation patterns observed through slope monitoring. The model calibration process has resolved that deformation patterns observed at Beauregard Landslide are best simulated using a model where (a) the basal shear surface geometry has been interpreted using a minimum curvature algorithm, (b) spatially variable residual friction is equal to 19° in the upper portion of the slope and 25° across the lower and central portion of the slide, and (c) landslide zones are discretely defined. These sophisticated three-dimensional simulations are required in order to account for the complexities inherent in a massive landslide.

Changing ground water conditions have been applied to the calibrated models addressing how slope stability is influenced by the development of a dewatering system and rising reservoir levels. The presumed drawdown achieved by an assumed drainage tunnel layout is effective in reducing deformation rates through the central and lower regions of the landslide. The upper slope is not influenced as water levels applied to numerical models remain constant through slope regions at higher elevations. These models also indicate that with slope drainage some increase in reservoir level may be achieved without exceeding deformation rates simulated under current water table conditions. However increasing the reservoir level to full capacity would not be acceptable even with the assumed drainage as slope displacement rates will accelerate beyond those achieved in simulation of current ground water levels. Based on these results, it is reasonable to
suggest further work in studying ground water conditions and the response of water
tables to drainage and reservoir filling scenarios be pursued in order to further refine
numerical models for testing slope response. A more detailed hydrogeology study would
be required to assess the effectiveness of a drainage tunnel in lowering water table
potential, and to refine the location and geometry of drainage infrastructure in order to
optimize its effectiveness. Ongoing studies would also benefit from ground water data
collected through the upper portion of the slope as water levels at higher elevations have
been inferred for this study. Further, if drainage construction and reservoir filling are
eventually carried out, slope monitoring will be critical for model verification.

8.9 Acknowledgments

The authors would like to thank G. Barla, M. Barla and G. Piovano for data access. This
work has been made possible through contributions by NSERC.

8.10 References

morphology, sampling density and interpolation methods on grid DEM accuracy.

Beauregard Landslide (Aosta Valley, Italy) using advanced and conventional techniques.
Submitted to Engineering Geology, November: 29 manuscript pages.


Kalenchuk K.S., Hutchinson, D.J. and Diederichs, M.S., 2010a. Morphological and geomechanical analysis of the Downie Slide using 3-dimensional numerical models: testing the influence of internal shears and interaction between landslide regions on
simulated slope behaviour. Submitted to Landslides manuscript # LASL 241: 32
manuscript pages.

Kalenchuk, K.S., Diederichs, M.S., Hutchinson, D.J. and Barla, G., 2010b. Integration of
two sources of slope monitoring data to characterize landslide behaviour, Beauregard
Landslide, Northwestern Italy. Submitted to Natural Hazards manuscript # NHAZ1308:
13 manuscript pages.

Kalenchuk, K.S., Diederichs, M.S. and Hutchinson, D.J., 2010c. Three-dimensional
numerical simulations of the Downie Slide to test the influence of shear surface geometry
and heterogeneous shear zone stiffness. Submitted to Computational Geosciences
manuscript # COM368: 24 manuscript pages.

Kalenchuk, K.S., Hutchinson D.J., Diederichs M.S., Barla G., Barla M. and Piovano G.,
2010d. Three-dimensional mixed continuum-discontinuum numerical simulation of the

Kalenchuk K.S., Hutchinson, D.J. and Diederichs, M.S., 2010e. Development and
calibration of numerical models for investigating trigger scenarios and mitigation
techniques for massive landslide hazard management. Proc. Geo2010, 63rd Canadian
Geotechnical Conference, Calgary, Alberta, Canada, September: 8 pp.

Lazzaro, D. and Montefusco, L.B., 2002. Radial basis function for the multivariate
interpolation of large scattered data sets. Journal of Computational and Applied
Mathematics 140: 521-536.


CHAPTER 9

Summary and Discussion

The management of massive, active landslide hazards is critical to the safety of the general public and engineered works such as hydro-electric operations. The ability to quantify significant deformation rates and understand the significance of changes to these rates requires expert understanding of large slope geomechanics, as well as intimate knowledge of site specific conditions including: geology, geomorphology, hydrogeology and slide behaviour. The research presented in this thesis contributes to the geomechanical knowledge base of massive landslides by providing methodologies for characterizing site conditions, analyzing measured slope behaviour, developing state-of-the-art numerical models, and interpreting model results.

9.1 Site Characterization

9.1.1 Three-Dimensional Interpretation of Shear Zone Geometry

Complex and geologically realistic, three-dimensional geometries of sub-surface shear interfaces can be interpreted using spatial statistics complemented by sound engineering judgment. The location of shear surfaces at specific points in space are identified in surface and subsurface data and with these data, fully three-dimensional shear zone
geometries can be defined using spatial prediction algorithms. Small- and large-scale geometric discrepancies result from the use of different spatial prediction techniques and geological assumptions of surface continuity, respectively. Different algorithms return different geometries and so multiple interpretations are possible for any given data set. The most appropriate algorithm for a given data set should be selected based on statistical comparison and assessment of spatial pattern reliability.

This methodology has been applied to both the Downie and Beauregard case studies. Each of these studies has different sub-surface data types available. Downie Slide has been sampled by numerous boreholes while Beauregard data is primarily from three seismic profiles. Despite the variation in data density and distribution, this methodology has performed well for both cases. Interestingly, it was found that the minimum curvature algorithm performed best, based on statistical goodness-of-fit, spatial pattern consistency and geological compatibility, for both sites. Numerical models with minimum curvature slip surface geometries returned the closest reproduction of measured slope deformation, indicating that these are most likely the truest three-dimensional interpretation of shear surface character.

These findings suggest that a minimum curvature algorithm is well suited to the interpolation of massive landslide shear zone geometries and that uniform data distribution is not required to produce reasonable results. This spatial prediction technique can therefore be applied to analysis of other landslides, for both back analysis of previous failures and interpretations of slopes which are currently active. Consider, as an example, the Vaiont Landslide; a rigorous, three-dimensional back analysis using the methodologies outlined in this thesis would be beneficial to the knowledge base of the
much debated mechanics of this well studied failure. With the definition of a reasonable shear surface geometry, efforts can be focused on better understanding other contributing mechanisms of instability.

In literature it is generally agreed that the basal slip surface at Vaiont followed bedding planes folded by a seat-shaped syncline. However, no consideration has been given to curvature or discontinuity of this failure surface. The exposed upper portion of the slope shows variability: the west is quite planar, the east appears to be more stepped and the central portion, in the Massalezza Valley shows undulating bedding plane surfaces. As demonstrated in this thesis, shear surface geometry is a principal factor controlling slope behaviour, and given this variation in the failure surface, a fully three-dimensional geometry is required for numerical simulation.

In order to undertake geometric interpretation, location data for the shear surface would be required. The upper portion of the Vaiont slip surface is exposed, making the collection of spatial data fairly straightforward. Conventional survey techniques may be used to gain point data across the exposed surface or more state-of-the-art approaches such as LiDAR or InSAR can be used to achieve full spatial coverage of the surface geometry. The position of the shear surface, below the landslide mass, may in part be resolved from pre-failure topographic data and geological mapping, and additional data could be collected by methods such as boreholes or geophysical seismic surveys.

The approach developed through this research is semi-automated and repeatable. The use of spatial prediction algorithms avoids the influence of pre-conceived ideas which may bias manual interpretations and contouring. However, a manual component to this approach is still necessary, because given the geological nature of this application, visual
inspection of pattern reliability is required. The geomechanical analysis of massive
landslides is improved with this new methodology for interpreting shear surface
geometries. Also, as demonstrated throughout this thesis, this approach to three-
dimensional interpretation can be applied to various data types, for example piezometric
records, displacement monitoring data and heterogeneous material properties.

9.1.2 Temporal Interpretations of Ground water Levels

Temporal changes to data density occur, when the spatial pattern of data collection differs
over time; a common occurrence for long term monitoring programs, as old instruments
are decommissioned and new ones are installed. The interpretation of ground water
levels is sensitive to this spatial-temporal variation in data density. When the distribution
of borehole piezometers changes, and water tables are incrementally interpolated over a
period of time, apparent ground water fluctuations are observed at close proximity to
decommissioned or installed instruments.

An approach to temporal data extrapolation has been developed in order to avoid the
apparent, and incorrect, interpretations of water table fluctuations. This technique is not
meant to be a detailed hydrogeological analysis, but rather it aims to generate reasonable
ground water states that can be applied to numerical models. The definition of changes to
ground water levels at Downie Slide, over different periods through the development and
operation of the Revelstoke Reservoir, has been refined. Drainage development lowered
ground water levels significantly through the central portion of the slide, at close
proximity to adits and boreholes, while no significant changes were observed across
regions of the slope more distal to drainage infrastructure. Ground water response to
reservoir filling was observed only by piezometers near the inundated landslide toe, and
small, gradual increases in piezometric levels have been observed over time through the central slope, near drainage infrastructure, as the boreholes lose capacity.

### 9.1.3 Heterogeneous Shear Zone Parameters

Variable deformation rates observed at different points on a landslide mass are in part controlled by the heterogeneous nature of shear zones. It has been demonstrated, through the Downie Slide case study, that numerical simulation results improve significantly when the stiffness of a shear zone is varied as a function of its thickness; thinner regions are stiffer than thicker regions. Also, material strength is thought to vary spatially, as is demonstrated by the Beauregard case history, where friction is interpreted to vary between the upper and lower portions of the slope.

It is intuitive to expect this type of heterogeneity, given the scale of these massive instabilities, and the fact that geological settings are very rarely homogenous. Any analysis of slope mechanics must therefore take into consideration the spatial variation in shear zone character. Consider again the Vaiont Landslide. The frictional strength of the slip surface has been widely debated; for example, Semenza and Ghirotti (2000) recognized montmorillonitic clay along the failure surface which would account for a very low friction angle (8° to 10°). Hendron and Patton (1969) summarize the multiple clay interbeds that are continuous over large areas of the sliding surface, and Broili (1967, in Müller 1987) suggested that failure occurred mainly in limestone and that clay was effectively non-existent, therefore the friction angle would not likely be less than 28°. It is probable that each author is in part correct, as observation of the slip surface at different points in space would likely contribute to completely different interpretations. A distribution of material strength at Vaiont might be resolved by detailed literature...
review, which should be complemented by a site visit, however; because the slip surface has been exposed for almost 50 years, its character has likely been altered, particularly with clay minerals which would be susceptible to weathering.

9.2 Interpreting Slope Deformation and Identifying Morphological Zones

Interpretations of overall landslide behaviour must consider not only the analysis of slope monitoring data, but also observations of morphology. Slope monitoring data returns spatially variable deformation rate magnitudes and displacement orientations. This complex behaviour is in part controlled by segregated failure mechanisms in different regions of a landslide mass. Morphological features are useful in recognizing these regions and identifying the mode of interaction between different zones, for instance the identification of zones of accumulation or depletion. Morphology can also provide some insight into the nature of slope deformation, for instance whether movements are deep-seated or surficial.

A number of movement zones have been recognized at Downie Slide and the slope has been discretized into morphological regions. The main slide mass has been delineated into upper, middle and lower regions. Different failure mechanisms within the main landslide body have been identified as retrogression in the upper slope, translation thought the central slope and rotational failure near the toe. The upper zone exhibits the highest, relative, rates, with slower deformation through the central slide and moderate rates nearer the toe. A number of secondary regions of instability have been identified, and the disconnect between surficial secondary instabilities and global behaviour of the
entire landslide body is recognized. This has resolved points of anomalous behaviour, for instance regions along the toe where high deformation rates, relative to those observed within the main slide body, are not reflective of overall slope displacements.

Interpretation of slope deformation patterns has also been completed for the Beauregard case study based on slope monitoring data provided to the author by Barla et al. (personal communications 2009). Data limitations, particularly in detailed morphological mapping and limited first hand observations due to accessibility challenges and timing restrictions, have not allowed the same level of detail in morphological zoning as was done at Downie Slide. As such, data analysis has been focused on defining global slope behaviour. The Beauregard Landslide shows high deformation rates through the upper portion of the slope and moderate rates near the southern toe. The central portion of the slope is slightly slower and movements observed at the north toe are effectively negligible.

This research has demonstrated that complex landslide mechanics are contributed to by different landslide zones exhibiting variable behaviour. Therefore to gain reasonable understanding of massive landslide global behaviour experts must recognize localized failure mechanisms and understand the contribution of these mechanisms to overall slope instability.

9.2.1 Improved Hazard Management by Understanding Composite Behaviour

An improved understanding of composite activity has significant implications for the interpretation of monitoring data. Recognition of primary and secondary instabilities is very important to the analysis of deformation data for slope hazard management. For
instance consider how surficial, secondary instabilities near the toe of Downie Slide exhibit higher deformation rates than those observed throughout the main landslide body. If these secondary deformation rates were misinterpreted as global behaviour, the entire landslide would be perceived to be moving much more rapidly than it truly is. Further, these secondary instabilities have been proven susceptible to changing boundary conditions that do not influence the entire landslide mass, for instance reservoir filling. Localized accelerations observed in surficial secondary regions are not necessarily cause for alarm; rather continued observation and an investigation into the trigger would be warranted.

9.2.2 Optimizing Slope Monitoring Practices

With an understanding of spatially discriminated behaviour, suggestions can be put forward to optimize ongoing slope monitoring practices. It is recognized that some areas of the slope are difficult to instrument due to limited accessibility, however sparsely sampled regions are a problem for interpreting data collected in monitoring programs. With knowledge of spatially discriminated slope behaviour, and identification of principal and secondary zones of instability, it is possible to target specific slope areas where new instrument installation would be most effective and thereby justify the added costs associated with difficult access. This includes regions where ongoing deformation makes significant contributions to the overall slope displacement patterns, for instance the upper region at Downie Slide, as well as areas that are most likely to respond to changing boundary conditions, for example the Downie Slide active zone. Areas with a less important contribution to the overall slide behaviour, for instance the lobe at Downie Slide, should be given lower priority for increased instrument density.
9.2.3 Deformation Patterns Common to Massive Slow Moving Landslides and Mechanisms of Rapid Failure

Downie and Beauregard have similar displacement patterns; the fastest relative rates are observed in retrogressive upper slope regions, slow movements occur through central portion, and moderate rates are achieved near the toe. This similarity reflects the common styles of activity between both landslides; composite translational, rotational and retrogressive. Both landslides also feature similarities in their adverse geological settings, for instance an unfavorably orientated foliation dipping into the valley.

It is interesting to point out that similar distribution of activity was observed at Vaiont prior to failure, the upper portion of the slope was moving faster than the lower portion (Müller 1987). This behaviour is also observed at La Clapière; the headscarp moves faster than lower portions of the slide (Casson et al. 2003, Jomard et al. 2007, Follacci 1987). Data and observations pertaining to spatially discriminated slope deformation rates are not available for Frank Slide or Goldau Slide.

This pattern of deformation common to massive landslides may, in part, be attributed to the state of failure through different portions of a slope. For instance, the shear zones near the toes of Downie and Beauregard landslides are well developed. At higher elevations there is evidence of later stage retrogressive failure. As continued down slope movement of the central and lower landslide regions progresses, the rockmass at higher elevations is debuttressed, progressive loss of cohesion occurs and a shear surface develops. The upper portions of the slopes, showing higher displacement rates, may then be effectively catching up with the lower landslide regions.
This process accounts for the material accumulation and spreading at the scree slopes at La Clapière. No accumulation zone has been observed at Downie Slide; it has been speculated that early landslide activity may have initially extended only as far as the boundary between the upper and central regions, leaving a zone of depletion which has since transitioned to a zone of accumulation as retrogressive blocks reach the lower landslide mass. Substantial tension cracks at Beauregard have been observed through the upper portion of the lower landslide region and it is hypothesized that these tension cracks may in fact be closing rather than propagating, however there is no field data to verify this hypothesis.

It is speculated that this process may be a key identifier of rapid failure potential. The onset of rapid failure may be attributed to two factors, first, the rate of toe displacement and second, the size of area where cohesion is abruptly lost. Consider that at Frank and Vaiont, creep at the toe was likely accelerated by coal mining and the hydrogeologic state respectively. This would have led to rapid debuttressing of the upper slopes, which in turn contributed to a larger-scale area experiencing abrupt loss in cohesion. Sudden failure of the upper slopes would have triggered rapid movement in the lower slopes by a number of potential mechanisms, such as: seismicity associated with mass movement, changes to pore pressure, or a bulldozing effect of a large impact between the upper and lower slope regions.

Now, consider Downie and Beauregard; in both cases, toe creep is gradual and as a result, retrogressive behavior in the upper slopes is observed as smaller-scale rockmass blocks breaking free, rather than sudden massive movement. This periodic degradation of the upper slope does not achieve the dynamic energy required to accelerate the lower slopes
into a state of rapid displacement. It is highly unlikely that Downie will ever transition to rapid massive failure, because the upper slopes have already developed shear surfaces, and so there is no likelihood of abrupt loss of total cohesion across a massive area. The likelihood of rapid massive failure at Beauregard is difficult to discuss without additional sub-surface data to characterize the state of shear surface development.

9.3 Development of Numerical Modelling Methodology

Literature review has demonstrated the need for sophisticated numerical methods for massive landslide simulation. Three-dimensional numerical models have been developed, utilizing 3DEC to reproduce observed slope deformation patterns. This contribution provides a methodology for the application of mixed continuum-discontinuum numerical methods to represent the landslide and undisturbed in situ material using deformable continuum blocks which interact along shear zones defined by discrete discontinuities. This approach to sophisticated modelling accounts for aspects such as; mesh density and grid size sensitivity testing, the inclusion of water tables, the buttressing load imposed by a reservoir, as well as boundary conditions and in situ stresses. The sophisticated numerical models developed for this thesis allow for complex three-dimensional slip surface geometry, heterogeneity in geomechanical shear zone properties, secondary shears within the landslide mass and mechanical interactions between discrete landslide zones.

9.4 Numerical Simulation of Observed Slide Behaviour

Three-dimensional numerical models have been calibrated to reproduce slope behaviour observed at the Downie and Beauregard landslides. The calibration process varies
between these two case studies because, due to site specific field investigation programs, each landslide has different information available:

- At Downie, an impressive number of boreholes have been cored and logged, providing information pertaining to the location of the shear zones and spatial variation in the thickness of the basal shear zone.

- At Beauregard, three seismic profiles have been completed helping to define the shear zone geometries. Only two boreholes have been cored and logged at Beauregard providing data too sparse to reasonably define variation in shear zone thickness, and no information is available pertaining to secondary shears within the landslide mass.

- Significantly more data from laboratory testing of material properties is available for this study from Beauregard than from Downie. This data provides a basis for assigning material properties to Beauregard models, and some understanding of how these properties vary spatially.

- Material properties for Downie have been derived from typical values for the geological conditions and model sensitivity testing.

- Landslide zones have been interpreted for both Downie and Beauregard. Zoning at Downie has been defined by studying spatial variations in morphological features and deformation processes. Zones at Beauregard are identified only by morphological features.
9.4.1 Application of Numerical Simulations to Landslide Hazard Management - Testing Mitigation Techniques

Once models are trained to reproduce observed slope deformation, mitigation techniques, such as slope drainage, can be numerically tested in order to explore how deformation rates may be slowed or eliminated. The refinement and proofing of remediation techniques is essential given the physical scale and apparent complexity of the massive instabilities. Forward modelling does not aim to define absolute values of changed deformation rates; rather, the goal is to assess which areas of the slope are most influenced, and to qualify if mitigation strategies will have a significant influence on deformation rates.

Changing ground water conditions have been applied to the calibrated models of the Beauregard Landslide to address how slope stability is influenced by the development of a dewatering system and changes to reservoir operating levels. These models have indicated that water table drawdown achieved by an assumed drainage system layout will be effective in reducing deformation rates through the central and lower regions of the landslide. The upper slope would not be influenced, as water levels applied to numerical modes remain constant through slope regions at higher elevations. These models also indicate that with slope drainage it would be possible to achieve some increase in the reservoir operating level. However, increasing the reservoir level to full capacity would not be acceptable, even with the assumed drainage, as slope displacement rates will accelerate beyond those achieved in simulation of current ground water levels.

By these results it is suggested that further work be done to study ground water conditions at Beauregard. A more detailed hydrogeological study is required to assess
the effectiveness of a drainage tunnel in lowering water table potential, and to refine the location and geometry of drainage infrastructure in order to optimize its effectiveness. Detailed hydrogeology can then be used to further refine numerical models for additional testing of slope response. Further, if drainage construction and reservoir filling are eventually carried out, slope monitoring will be critical for model verification.

9.4.2 Application of Numerical Simulations to Landslide Hazard Management - Testing Trigger Scenarios

Landslide monitoring practices provide information pertaining to surficial movements, sub-surface deformation, piezometric variations and, in some cases, seismicity. The interpretation of complex monitoring data is difficult and meticulous; expert understanding of landslide mechanics and experience in resolving data from sensor networks is necessary for effective hazard management. The interpretation of instrument data for massive landslide risk assessment is complicated by the challenge of defining a rate of movement or a magnitude of displacement that is indicative of near or immediate mass movement.

Numerical modelling can be used to simulate slope response to specific scenarios. In doing this it is possible to test for changes in rate that are significant indicators of imminent rapid failure. Numerical modelling can be used to virtually observe individual sensors as well as the collective response of an instrumentation network to different trigger scenarios. This style of testing can contribute to the establishment of site specific trigger thresholds for risk management. Further, synthetic data records of this sort can train decision support systems to recognize significant combinations of sensor output indicative of slope behaviour that would be of concern. This has valuable application in
hazard management of complex landslide systems by enhancing the ability of decision makers to respond to large amounts of data, rapid changes in a system and highly complex model dynamics.

Forward testing of potential trigger scenarios has been completed for Downie Slide. Rapid reservoir drawdown and complete loss of drainage capacity has demonstrated the application of sophisticated numerical simulations to hazard management. Negligible changes to global landslide behaviour occurred for the simulation of rapid reservoir drawdown, however it is notable that drawdown is likely to have a more significant impact on localized surficial instabilities at the landslide toe, and the compounded influence of drawdown and accelerate toe unloading has not been accounted for. Slope accelerations occur in response to simulated total loss in drainage capacity. The magnitude of the rate increases are the same as the magnitude of decelerations observed during simulated drainage development. While the total failure of the drainage system is extremely unlikely, particularly with continued site maintenance, this worst case scenario would only cause minor slope accelerations.

9.5 References

Barla, G., Barla, M. and Piovano, G., 2009 personal communication


CHAPTER 10

Conclusions

This research has improved understanding of the physical nature and ongoing behaviour of the Downie and Beauregard landslides, and in doing so has improved the knowledge base of massive landslide geomechanics. Assessment of landslide mechanics draws from comprehension of the geological, morphological, hydrogeological and geomechanical setting and observations of slide behaviour through slope monitoring. Key components influencing slope deformation have been investigated through numerical modelling. The following sections summarize the main conclusions made specific to the case studies, as well as to the analysis of massive slopes in general.

10.1 Downie Slide

10.1.1 Landslide Site Characterization

Early recognition of landslide morphological zones at Downie Slide by Piteau et al. (1978) and Patton and Hodge (1975) identified a head area immediately down slope from the head scarp, a central area between the head and the toe of the slide, the south knob area located at the downstream toe, the north knob area located at the upstream toe and an
active area located between the south and north knob areas. These regions were largely interpreted from aerial photographs and their boundaries were not precisely delineated.

The research presented in this thesis has concluded that the modern Downie Slide is classified as a massive, active, composite, extremely slow rockslide, and a new interpretation of morphological zones at Downie Slide has been developed. These zones have been interpreted from morphological features identified in airborne LiDAR data, observations made during site visits by the author in 2008 and 2009, and by thorough analysis of slope monitoring data revealing spatially discriminated failure mechanisms and deformation patterns:

- The main slide body is made up of the upper, middle and lower regions.
- Secondary instabilities include the talus slopes, the north disturbed zone, the lobe, the basin, the over-steepened slopes, the toe slump and the toe slough regions.
- The north knob is an inactive area.

A review of piezometric data has confirmed that:

- Drainage development lowered ground water levels significantly in slide areas proximal to adits and boreholes, while portions of the slope more distal to drainage infrastructure saw no significant changes.
- The ground water response to reservoir filling was observed only by piezometers in close proximity to the inundated landslide toe.
- Since reservoir filling small increases in piezometric levels have been observed in portions of the landslide near drainage infrastructure due to gradual deterioration of the drainage system.
10.1.2 Slope Behaviour

Slope monitoring has been ongoing at Downie Slide for more than 35 years providing a substantial record of slope displacement and ground water data. Some work has been done by BC Hydro to identify changes in deformation rates in response to major changes to ground water boundary conditions (i.e. drainage system development and reservoir filling), however, in the literature, there has been no comprehensive analysis of spatial and temporal variations in slope displacements.

This thesis presents an interpretation of deformation at Downie Slide based on survey monitoring and inclinometer data. It has been found that Downie Slide does not behave as a monolithic instability, rather, different landslide zones display specific landslide processes:

- The upper slope features retrogressive failure and this region displays the fastest rates observed on the main landslide body.
- Translational sliding occurs through the middle region, however displacements are effectively negligible.
- The lower zone exhibits translational and rotational sliding. The active landslide zone is found within the north lower region, and along the reservoir the lower region features extensive toe sloughing.
- The talus slopes have accumulated from gradual ravelling of the side and head scarps.
The north disturbed zone is hypothesized to have initiated in response to the main instability to its south. Active deformation is believed to be surficial; however this is difficult to conclude without sub-surface deformation data.

The lobe is a secondary failure, surficial movement is active today.

The north knob is inactive and makes no contribution to modern, global landslide behaviour.

It is hypothesized that material from the basin has gradually displaced toward the active area, however there are no survey monuments present to provide magnitude and direction of modern, local deformation.

The over-steepened slopes show ongoing surficial deformation.

The toe slump is a localized instability with anomalously high deformation rates (yet is still classified as very slow) which are not representative of the overall slide behaviour.

It has also been concluded that global slope behaviour can be described as extremely slow moving; relatively high rates of movement occur in the upper slope region, moderate rates are observed through the central toe of the slide, while the middle portion of the slope is characterized by lower velocities.

Research findings resolve that landslide hydrogeology is an important boundary condition influencing the modern behaviour of Downie Slide. From the analysis of temporal changes in landslide behaviour it is clear that different regions of the slope respond differently to changing boundary conditions:
• Deformation in the upper region is localized and not responsive to major changes in ground water levels (drainage development, reservoir filling and gradual losses in drainage capacity).

• The central and lower portions of the slope show decelerations in response to drainage development, no significant response to reservoir filling, and minor accelerations due to gradual losses in drainage capacity.

• Surficial, secondary failures at the landslide toe show negligible change in response to drainage system development and over the operating life of the reservoir. These surficial features exhibited a more significant response to reservoir filling as accelerations were observed through the over-steepened slope, toe slump and toe slough zones.

10.1.3 Numerical Modelling

Previous analyses of the Downie Slide have utilized two-dimensional methods and assumed a monolithic mass with heterogeneous material properties and a singular water table (for example: Enegren 1995, Kjelland 2004). Sophisticated three-dimensional numerical simulations of Downie Slide have improved understanding of the landslide geomechical characteristics. Through numerical studies we now know that:

• The basal slip surface geometry is best interpreted using a stepped minimum curvature algorithm. This is considered to be geologically realistic, however the depth to which the sub-vertical face (along the south landslide boundary) extends below the topographic surface is unknown. This three-dimensional interpretation is likely closest to reality as it performs best when numerically modelled.
Continuous minimum curvature and krigging algorithms also perform reasonably well.

- The basal shear zone at Downie Slide varies considerably in thickness and the stiffness of this zone varies as a function of that thickness. Thinner areas are stiffer while thicker areas are less stiff. Numerical simulations prove to be sensitive to spatial heterogeneity in shear zone stiffness; however the absolute scale of stiffness variability is not easily defined.

- Secondary internal shears make minor contributions to global landslide behaviour. These results are not surprising as inclinometer data shows that internal shears do not discretely contribute to a significant portion of the observed slope deformation.

- Important mechanical interactions occur between landslide morphological zones, for example it is apparent that the main slide body receives some load from adjacent secondary instabilities such as the talus slopes, the basin and the north disturbed zone.

Numerical simulation of changing ground water conditions produces reasonable correlations with field observations, particularly for those changes to ground water boundary conditions that influence the global activity of Downie Slide:

- Simulated drawdown by drainage development slowed overall landslide deformation rates.

- Reservoir filling resulted in accelerations through the lower portion of the slope.
• Gradual losses in drainage capacity over the reservoir operating life had negligible influence on model global behaviour with the exception of minor accelerations near the toe in the active region.

• The analysis of water table data and slope response combined with observation of numerical models has brought to light how different morphological regions respond differently to changing boundary conditions.

Understanding why and where numerical simulations correlate with or deviate from field observations enables experts to forward test potential trigger scenarios and the effectiveness of engineered mitigation approaches. Forward testing of potential trigger scenarios has demonstrated the application of these sophisticated numerical simulations to hazard management:

• Negligible change in simulated behaviour occurred in response to rapid reservoir drawdown. However, it is noted that simulations represent global behaviour and secondary, surficial instabilities at the landslide toe would likely have a more significant response. The compounded influence of drawdown and accelerated toe unloading has not been accounted for in this study.

• Total loss in drainage capacity has resulted in accelerations of the same magnitude as those decelerations observed during simulated drainage development. While the total failure of the drainage system is extremely unlikely, particularly with continued site maintenance, simulations indicate that this worst case scenario would only cause minor slope accelerations and deformation rates would be comparable to those observed prior to drainage development.
10.2 Beauregard Landslide

10.2.1 Landslide Site Characterization

The Beauregard Landslide can be classified as massive, active, composite, extremely slow rockslide according to the Cruden and Varnes (1996) classification scheme. Beauregard site conditions have been reviewed for this thesis from work by Barla et al. (2006, personal communication 2009, personal communication 2010) and Miller et al. (2008).

10.2.2 Slope Behaviour

Previous interpretations of Beauregard slope behaviour using conventional survey techniques (Barla et al. 2006, Miller et al. 2008) have characterized the landslide toe as the most active region, and in recent years GBInSAR data has revealed a movement zone at higher elevations (Barla et al. 2010). Through this research global deformation at Beauregard Landslide has been analyzed based on the amalgamation of displacement data collected by Leica surveys, GPS surveys and GBInSAR. From these data it has been concluded that:

- The highest deformation rates occur through the upper portion of the slope where behaviour is predominantly retrogressive and translational.

- Moderate rates, relative to the global average, are observed near the southern toe and the central portion of the slope is slightly slower. Translational sliding occurs through the central and lower regions transitioning to rotational sliding at the landslide toe.
• Negligible movements are observed at the north toe.

10.2.3 Numerical Modelling

Numerical simulation of the Beauregard Landslide has previously been limited to two-dimensional continuum methods which indicated by shear strength reduction techniques that the lower portion of the slope was most vulnerable to instability (Barla et al. 2006). Through numerical studies of continuous deformation at Beauregard we now know that:

• Observed deformation patterns are best simulated when the basal shear surface geometry is interpreted using a continuous minimum curvature algorithm.

• The frictional strength of shear surfaces varies spatially. In the upper portion of the slope behaviour is assumed to be brittle and the lower portion of the slope is presumably at residual strength. Observed slope behaviour is best reproduced when the post-peak frictional strengths are 19° and 25° for the upper and lower regions of the landslide respectively. These values are comparable to laboratory test results.

• Differential movements at Beauregard Landslide are best simulated by allowing landslide zones to displace independent of one another rather than simulating the entire slope as a monolithic mass.

Forward testing of the effectiveness of slope drainage as an engineered mitigation approach has been completed, and from the results it is apparent that:

• The development of a slope drainage system with current reservoir levels would slow deformation rates across the lower and middle regions of the landslide. The
most significant changes would be observed across the landslide toe, and no change would occur at higher elevations.

- With slope dewatering some increase to the Beauregard Reservoir level can be achieved safely, however filling to full capacity accelerates slope rates to unacceptable levels, as they exceed those modelled with current water level conditions.

10.3 Geomechanical Numerical Modelling of Massive Landslides

Literature review has demonstrated that few contributions have been made to complex numerical simulation of ongoing activity in slow moving slopes. Numerical landslide studies commonly utilize two-dimensional methods and are generally focused on singular, or limited numbers of, factors contributing to slope mechanics. Numerical simulations must be able to account for those main geomechanical factors controlling behaviour in order to adequately reproduce observed deformation patterns. Multiple factors must be tested in order to identify which are controlling, and when a number of factors are identified as influential, models should incorporate all of them.

Three-dimensional numerical methods developed through this research improve state-of-the-art landslide modelling as they are capable of accounting for improved interpretations of irregular geometric configurations, heterogeneous geological setting, complex hydrogeology and composite landslide systems. The following conclusions have been drawn regarding modelling procedures for massive landslide simulation:

- The three-dimensional geometry of basal slip surfaces is a major controlling factor in landslide deformation patterns. In order to reasonably reproduce observed
slope behaviour, fully three-dimensional, geologically realistic, shear surface geometries must be simulated.

- Heterogeneity in shear zone strength parameters should be accounted for when data is available.

- Shear zones have some magnitude of thickness and when they are numerically replicated as discrete joint elements with zero thickness the thickness discrepancy will influence model mechanics. Stiffness values must be assigned to account for this difference; in this study shear and normal stiffness values are in the order of 50 and 100 MPa/m, respectively. Where data pertaining to spatial variation in shear zone thickness is available, heterogeneous stiffness parameters should be defined accordingly.

- Complex shear surface geometry and heterogeneous shear zone strength parameters contribute to localized stress concentrations and joint slip; adding to intricate patterns of three-dimensional slope deformation.

- Complex shear surface geometry and heterogeneous shear zone strength parameters assigned to models which closely reproduce measured slope deformation may be taken as the truest three-dimensional interpretation of shear surface character.

- Secondary shears should be numerically represented where they are justified by morphological zoning. When secondary shears are not simulated, a low deformation modulus can achieve reasonable results by allowing internal strains to be adequately accommodated by deformation within the simulated landslide body.
The influence of secondary shears should be tested on a case-by-case basis as the role of internal structure varies for specific case studies.

- Morphological features must be studied to recognize landslide zones; these should be discretely defined in numerical simulation to allow for mechanical interaction between regions.

- This approach to mixed continuum-discontinuum landslide modelling reproduces global deformation patterns rather than localized activity.

- The numerical modelling presented in this thesis has contributed to improved understanding of key geomechanical factors controlling massive instabilities.

10.4 Limitations of Presented Research

The research presented has been limited in part by the data collection practices. The distribution of displacement monitoring instruments at Downie Slide has left some regions of the slope sparsely sampled, such as the upper region, the basin, and the disturbed north zone. Also, sparse ground water sampling through the upper regions of both landslides has meant that water tables at high elevations are largely inferred. These limitations were unavoidable as the development of monitoring networks and the collection of data was initiated well before the commencement of this research. Recommendations have been made in this thesis for future instrument installations. The high cost of developing further instrumentation requires careful consideration.

There is some limitation to the numerical modelling presented in this thesis associated with the application of dynamic ground water conditions. Changes to ground water levels in numerical simulations are instantaneous, and in reality these changes occur over some
period of time. Improvements to simulated behaviour in numerical models may be achieved with the application of staged or gradual changes in ground water levels. Also, in reality, ground water levels influence a number of factors; such as: hydrostatic pressures in the rockmass, fracture networks and on shear surfaces, as well as rockmass density and dilatancy; however in these numerical models the influence of changing ground water levels is limited to pore pressure in the rockmass and acting on shear surfaces.

Another shortfall to numerical modelling is the continuum nature of the landslide mass, with the exception of locations where multiple shears or discrete regions are defined. If smaller-scale fracture networks were simulated, or the propagation of fractures within the model were possible, numerical simulations may be more capable of capturing localized slope behaviour. This limitation is very difficult to overcome without exceedingly detailed structural mapping during site investigation. Further, computer resource limitations would likely constrain the resolution of applied fracture networks.

10.5 Recommendations and Future Areas of Research

This thesis has demonstrated the need for sophisticated three-dimensional numerical simulation of complex landslide systems in order to adequately reproduce observed slope deformation. Trained models can be used for testing of trigger scenarios and engineered mitigation techniques, thus providing significant value to massive landslide hazard management. Recommendations to future research focus on further contributions to state-of-the-art landslide modelling.
The research presented in this thesis looks at long term landslide behaviour. It would be interesting to simulate seasonal variation in slope deformation by applying seasonal fluctuation in ground water levels to numerical models. Unfortunately field data is not collected frequently enough at Downie Slide to analyze seasonal variation in deformation rates. Therefore, this avenue of modelling could not be verified without increasing slope monitoring frequencies. Seasonal behaviour at Beauregard has been studied and slope response to meteoric and piezometric conditions is well documented (Miller et al. 2008) providing a solid base for comparison to numerical simulations.

Numerical models presented in this thesis reproduce global landslide behaviour. A significant advancement to three-dimensional landslide modelling could be gained by incorporating fracture propagation. This advance would enable two areas of research to be pursued; the evolution of slide zones and the simulation of localized behaviour. This added complexity to numerical modelling would require more detailed site investigation to refine spatial heterogeneity in geology and structure, as the natural development of landslide zones and spatially discriminated failure mechanisms are contributed to by localized rockmass conditions as well as slope dynamics.

10.6 Contributions

The research completed for this thesis has been published or submitted for publication as seven peer-reviewed international journal publications. These contributions to landslide geomechanics, numerical modelling, and landslide hazard management have resulted in a multi-faceted thesis covering a wide variety of topics.
10.6.1 Articles Published in Refereed Journals


10.6.2 Articles Submitted to Refereed Journals


10.6.3 Invited Book Chapters


10.6.4 Invited Papers


10.6.5 Fully Refereed Conference Papers (Papers Reviewed and Published in Proceedings)


**10.6.6 Partially Refereed Conference Papers (Abstract Reviewed – Paper Published in Proceedings)**


**10.6.7 Published Abstracts**

of a large, complex slowly moving landslide. Geophysical Research Abstracts Vol. 9, 05871, European Geosciences Union.

**10.6.8 Posters and Presentations**


10.7 References


Appendix A

Predicted geometries for continuous shear surfaces

This appendix includes figures illustrating continuous shear surface geometries for all spatial prediction algorithms (excluding those illustrated in Figure 2.7).
Figure A.1: Continuous three-dimensional geometries (looking northwest) interpreted using the minimum curvature algorithm with varying internal and boundary tensions and universal krigging.
Figure A.2: Continuous three-dimensional geometries (looking northwest) interpreted using radial basis functions and a moving average algorithm.
Figure A.3: Continuous three-dimensional geometries (looking northwest) interpreted using inverse distance to a power functions and linear triangulation.
Figure A.4: Continuous three-dimensional geometries (looking northwest) interpreted using low-order polynomials, a natural neighbour algorithm and a nearest neighbour algorithm.
APPENDIX B

3DEC Modelling

B.1 Model Theory

This section overviews modelling theory found in the 3DEC Theory and Background Manual (Itasca 2003) which has not already been included in the thesis manuscript chapters.

B.1.1 Block Interaction

3DEC recognizes contacts using a cell mapping routine which identifies adjacent blocks by enveloping each block within a search perimeter and checking for contacts with all other blocks mapped within that perimeter. A minimum vertex displacement tolerance controls the frequency of remapping and contact testing as blocks move. This tolerance parameter is also used for contact recognition; when the distance between adjacent blocks is less than the tolerance, a contact is created. If a contact is created, but the blocks are not actually touching, the contact will carry no load. When new contacts are recognized, the physical contact between adjacent blocks is numerically replicated as a data element describing contact information such as strength parameters, the common plane unit normal and active forces. 3DEC contacts are soft, meaning that the measurable contact stiffness is represented as finite normal stiffness. Hard contacts, in contrast, use algorithms to avoid block penetration.
The common plane defined for each contact describes the orientation and movement at each time step. Contact type (i.e. face-face, edge-face, vertex-vertex, etc.) is assigned based on the number of vertices on each block touching the common plane. Type determines the mechanical response at a contact; for instance stress, rather than force, would contribute to face-to-face behaviour. Block faces at a contact are discretized into sub-contacts, which are created for each surface node on a deformable block. Sub-contacts are used to track contact conditions such as forces, sliding and block separation. Sub-contacts on opposing faces calculate relative displacements and forces using contact logic described by a set of parallel springs.

### B.1.2 Sub-contact Force Update

The relative velocity \( V_i \) across a sub-contact is obtained from the sub-contact velocity and the velocity of the opposite face. Incremental displacements \( \Delta U_i \) over time \( t \) are then:

\[
\Delta U_i = V_i \Delta t \tag{A1}
\]

which can be resolved into normal and shear components according to the sub-contact unit normal. All sub-contacts on a common face are assigned a common unit normal taken as the unit normal to the common plane. Incremental displacements are used to calculate the normal and shear elastic force increments:

\[
\Delta F_i^n = -K_n \Delta U_i^n A_c \tag{A2}
\]

\[
\Delta F_i^s = -K_s \Delta U_i^s A_c \tag{A3}
\]
where $K_n$ and $K_s$ are normal and shear stiffness, respectively, and $A_c$ is the sub-contact area. Elastic force increments are adjusted according to contact constitutive relations and then are used to update the contact total force vector. In deformable blocks, forces are resolved at vertices and added to other gridpoint forces.

**B.1.3 Continuum Behaviour**

Discrete blocks in 3DEC are made deformable by discretizing individual blocks into constant strain-rate elements of tetrahedral shape. Tetrahedral elements are used because when mesh zones have more than 4 gridpoints, combinations of gridpoint displacements can produce situations of no strain, where no opposing forces, concentrated at mesh gridpoints, will develop, resulting in unopposed deformation of alternating direction. Lagrangian analysis is utilized in 3DEC; the grid is able to move and deform as incremental displacements are added to the gridpoint coordinates.

Discretized blocks create a three-dimensional finite difference model which explicitly solves a system of differential equations. The set of differential equations relate the mechanical and kinematic behaviour of the system by transforming the laws of motion into a discrete form of Newton’s laws at the gridpoints. The explicit approach to solving equations of motion is ideal for simulating large strain and physical instability, where the implicit matrix solution scheme would be numerically unstable. At each time step, the equations of motion are solved at each gridpoint. An equivalent static problem is derived at each time step by applying an inertial term to the equations of motion to approximate an equilibrium solution. Given the local state of stress, active forces, velocities and displacements are determined for each gridpoint. Strain rates are subsequently derived and the stress state is then updated using the stress increment defined for that given time.
interval by the constitutive equations. Given the stress state at time $t$ and the total strain increment for a time step, $\Delta t$, the corresponding stress increment and the new stress state can be determined at time $t+\Delta t$. When plastic deformations are involved, only the elastic part of the strain increment will contribute to the stress increment, therefore a correction is required for the elastic portion of the stress increment computed from the total strain increment in order to obtain the actual stress state for the next time step.

For deformable blocks, the equations of motion are applied to the gridpoints at each time step. A net gridpoint force vector is calculated, accounting for (1) the body forces due to gravity and (2) the gridpoint forces due to all the external loads acting on a gridpoint, the stresses within all zones adjacent to the gridpoint and, where the gridpoint is at a block boundary, the sub-contact forces. When a body is not at either steady state flow or equilibrium, the body will be accelerated according to the finite difference form of Newton’s second law of motion. Gridpoint displacements are then related to strains and rotations, constitutive relations are used to define a new state of stress, and gridpoint forces are updated for the next time step.

**B.2 Model Code**

This section provides the 3DEC and FISH code developed for this research (significant contributions to the development of this code have been provided by C. Carranza-Torres). The programming files provided here are sequential as they are called by 3DEC and sample data files are include to demonstrate file format. The template files provided here are not replicates of the exact files used in this thesis as values and file form vary.
depending on specific run requirements. All runs have been saved electronically and submitted to Dr. Mark Diederichs for archive.

**B.2.1 Master File**

This master file calls the varying files over the modelling sequence.

`Master.dat`

```dat
new
call stage1_build_geom.dat
call stage2_cut_shears.dat
call stage3_mesh_generator.dat
call stage4_load_instruments.dat
call stage5_rockmass_pp.dat
call stage6_set-up_sequence.dat
call time_steps.dat
```

**B.2.2 Build Geometry**

This file loads the topographic surface information to build the model topographic geometry.

`stage1_build_geom.dat`

```dat
set ismax 5000000 ; <-- for visualization purposes...

DEF _general_variables
  size_of_grid = 100
  _fname_topo_model_INFO = 'ELEV_XYZ_[100]_[3DEC_info].txt'
  _fname_tab_TOPOGRAPHY = 'TOPOGRAPHY_[3DEC_tables].txt'
  _ntable_TOPOGRAPHY = 1000
END _general_variables

; BEGIN: load surface topography information...
call ELEV_XYZ_[100]_[3DEC_geom].dat

DEF _read_header_info_topography_blocks
  array _aa(100)
  IO_WRITE = 1
  IO_READ = 0
  IO_ASCII = 1
  status=open(_fname_topo_model_INFO,IO_READ,IO_ASCII)
  status=read(_aa,1) ; <-- ignore the 1st line...
  status=read(_aa,1) ; <-- ignore the 2nd line...
  status=read(_aa,1) ; <-- number lines in the file
  _nlines = int(parse(_aa(1),1))
```
status=read(_aa,1)
_xEast_MIN = float(parse(_aa(1),1))
status=read(_aa,1)
_xEast_MAX = float(parse(_aa(1),1))
status=read(_aa,1)
_zNorth_MIN = float(parse(_aa(1),1))
status=read(_aa,1)
_zNorth_MAX = float(parse(_aa(1),1))
status=read(_aa,1)
_yUp_MIN = float(parse(_aa(1),1))
status=read(_aa,1)
_yUp_MAX = float(parse(_aa(1),1))
status=read(_aa,1)
_nblocks_xEast = int(parse(_aa(1),1))
status=read(_aa,1)
_nblocks_zNorth = int(parse(_aa(1),1))
status=read(_aa,1)
_npts_xEast = int(parse(_aa(1),1))
status=read(_aa,1)
_npts_zNorth = int(parse(_aa(1),1))
status=read(_aa,1)

_DEF _read_coords_info_topography_blocks
; ---- attention with maximum number specified for arrays...
array _xEast_3DEC_block(300,300)
array _zNorth_3DEC_block(300,300)
array _yUpward_3DEC_block(300,300)
IO_WRITE = 1
IO_READ = 0
IO_ASCII = 1
status=open(_fname_topo_model_INFO,IO_READ,IO_ASCII)
status=read(_aa,15)
loop i (1,_npts_xEast)
  loop j (1,_npts_zNorth)
    _status = read(_aa,1)
    _xEast = float(parse(_aa(1),1))
    _zNorth = float(parse(_aa(1),2))
    _ymin = float(parse(_aa(1),3))
    _ymax = float(parse(_aa(1),4))
    _xEast_3DEC_block(i,j) = _xEast
    _zNorth_3DEC_block(i,j) = _zNorth
    _yUp_MIN = _ymin
    _yUpward_3DEC_block(i,j) = _ymax
  end_loop
end_loop
status=close
END

_DEF _read_header_info_topography_blocks
_DEF _read_coords_info_topography_blocks

save stage1.sav
B.2.3 Cut Shear Interfaces

This file cuts shear zones geometries into the topographic model

stage2_cut_shears.dat

res stage1.sav
set atol 0.05 ; <--- do not change: to be able to cut 2nd shear zone...

; load INTERFACE information... this varies depending n basal slip surface geometry
call lower_shear_geom_tables.txt ; secondary continuous shear
; load TOPOGRAPHY information...
call TOPO_multiquad_[3DEC_tables].txt

DEF _interpolate_surface
_flag_hit_outsider = int(0)
_n_scanlines = int(table_size(_table_num))
_min_Z_scanlines = ytable(_table_num,1)
_max_Z_scanlines = ytable(_table_num,n_scanlines)
_npoints_per_scanline = int(table_size(_table_num+1))
_min_X_scanlines = xtable(_table_num+1,1)
_max_X_scanlines = xtable(_table_num+1,npoints_per_scanline)
if _x_surf < _min_X_scanlines
   _dummy = out('Error: _x_surf < _min_X_scanlines...')
   command
   pause
   end_command
end_if
if _x_surf > _max_X_scanlines
   _dummy = out('Error: _x_surf > _max_X_scanlines...')
   command
   pause
   end_command
end_if
if _z_surf < _min_Z_scanlines
   _dummy = out('Error: _z_surf < _min_Z_scanlines...')
   command
   pause
   end_command
end_if
if _z_surf > _max_Z_scanlines
   _dummy = out('Error: _z_surf > _max_Z_scanlines...')
   command
   pause
   end_command
end_if

; ----- determine ntab_LOWER and ntab_UPPER ----- 
_flag_found_interval = 0
loop i (1,n_scanlines-1)
   if _flag_found_interval = 0
      _zLOWER = float(ytable(_table_num,i))
      _zUPPER = float(ytable(_table_num,i+1))
      if _zLOWER = _key_outsider
         _flag_found_interval = 1
         _zLOWER = float(ytable(_table_num,i))
         _zUPPER = float(ytable(_table_num,i+1))
      end_if
   end_if
end_loop

356
_flag_hit_outsider = 1
end_if
if _zUPPER = _key_outsider
    _flag_hit_outsider = 1
end_if
if _z_surf <= _zUPPER
    if _z_surf >= _zLOWER
        _ntab_LOWER = int(xtable(_table_num,i))
        _ntab_UPPER = int(xtable(_table_num,i+1))
        _flag_found_interval = 1
    end_if
end_if
end_if
end_loop
if _flag_found_interval = 0
    _dummy = out('Error: problems finding interval...')
command
    pause
end_command
end_if
; ----- determine values of _yLOWER and _yUPPER based on value of _z_surf,
; consulting tables _ntab_LOWER and _ntab_UPPER -----
_yLOWER = table(_ntab_LOWER,_x_surf)
_yUPPER = table(_ntab_UPPER,_x_surf)
if _yLOWER = _key_outsider
    _flag_hit_outsider = 1
end_if
if _yUPPER = _key_outsider
    _flag_hit_outsider = 1
end_if
_y_surf = _yLOWER+(_yUPPER-_yLOWER)*(_z_surf-float(_zLOWER))/(float(_zUPPER)-
    float(_zLOWER))
END

; Functions to compute dd and dip from coordinates of 3 points...
DEF _acos_x ; <-- arcos function...
    if _x = 1.0
        _acos_x = 0.0
    else
        _acos_x = 0.5*pi-atan(_x/sqrt(1-_x^2))
    end_if
END

DEF _function_get_dd_and_dip
    ; Attention: direction 1 is the 'x' direction in 3DEC
    ; direction 2 is the 'y' direction in 3DEC
    ; direction 3 is the 'z' direction in 3DEC
    ; dd is measured as in 3DEC (positive is clockwise from 'y' 3DEC direction)
    ; dip is measured as in 3DEC (positive is down the horizontal plane)
    ; Points A, B and C should be entered in a way consistent with DOT PRODUCT
    ; definition, normal to plane is = vector AC x vector BC
    ; Note: the function atan2(y,x) in 3DEC is the aractan function of y/x and
    ; returns an angle in the range [pi,-pi]
    _min_n12 = 0.001; <-- small value to assign to (1) and (2) coordinates in case of normal vector
    ; coincident to (3) direction (to avoid ambiguity in the definition of dd in the case of horizontal
    ; plane in 3DEC)
; ---- compute vector vA ----
_vA_1 = _pA_1 - _pC_1
_vA_2 = _pA_2 - _pC_2
_vA_3 = _pA_3 - _pC_3
; ---- compute vector vB ----
_vB_1 = _pB_1 - _pC_1
_vB_2 = _pB_2 - _pC_2
_vB_3 = _pB_3 - _pC_3
; ---- compute (unit) normal vector vN ----
_temp1 = _vA_2*_vB_3 - _vA_3*_vB_2
_temp2 = _vA_3*_vB_1 - _vA_1*_vB_3
_temp3 = _vA_1*_vB_2 - _vA_2*_vB_1
_vN_1 = _temp1 / sqrt(_temp1*_temp1+_temp2*_temp2+_temp3*_temp3)
_vN_2 = _temp2 / sqrt(_temp1*_temp1+_temp2*_temp2+_temp3*_temp3)
_vN_3 = _temp3 / sqrt(_temp1*_temp1+_temp2*_temp2+_temp3*_temp3)
; Do not allow normal vector parallel to the (3) direction... (to avoid ambiguity in the definition of dd)
if _vN_1 = 0
if _vN_2 = 0
    _vN_1 = _min_n12
    _vN_2 = _min_n12
end_if
end_if
; Compute (unit) horizontal vector vH coincident with the strike direction...
_vH_1 = -_vN_2/sqrt(_vN_1^2+_vN_2^2)
_vH_2 = _vN_1/sqrt(_vN_1^2+_vN_2^2)
_vH_3 = 0.0
; Define the OUTPUT variable 'dd' by measuring the angle that vector vH forms with the direction (2)
_angle = atan2(_vH_2,_vH_1);
if _angle <= 0
    _angle = 2*pi+ _angle
end_if
if 0.5*pi- _angle < 0
    _sdir = 2.5*pi- _angle ; <-- '_sdir' is the strike direction...
else
    _sdir = 0.5*pi- _angle ; <-- '_sdir' is the strike direction...
endif
if _sdir > 1.5*pi
    _dd = _sdir - 1.5*pi
else
    _dd = _sdir + 0.5*pi
end_if
_dd_deg = _dd * 180.0/pi
; Compute (unit) dip vector vDH perpendicular to 'strike vector' vH and
; perpendicular to 'normal vector' vDH ...
_vDH_1 = -vH_2/sqrt(vH_1^2+vH_2^2)
_vDH_2 = -vH_1/sqrt(vH_1^2+vH_2^2)
_vDH_3 = 0.0
; Compute OUTPUT variable 'dip' by measuring angle between vectors vDH and vN...
_cos_THETA = vDH_1* vN_1 + vDH_2* vN_2 + vDH_3* vN_3
_x = _cos_THETA
_THETA = _acos_x ; <-- call the function arcos...
_dip = 0.5*pi- _THETA
_dip_deg = _dip * 180/pi
END

358
DEF _cut_shear_INTERFACE

__ntable_INTERFACE = int(__ntable_INTERFACE)
__nregion_above_INTERFACE = __nregion_above_INTERFACE
__yMIN = __yUp_MIN

loop ii (1, __npts_xEast-1)
  loop jj (1, __npts_zNorth-1)
    _key_outsider = 10000
    _flag_outsider_TRIAG_1 = 0
    _flag_outsider_TRIAG_2 = 0
    _flag_outsider_TRIAG_3 = 0
    _flag_outsider_TRIAG_4 = 0
    _B00_1 = __xEast_3DEC_block(ii,jj)
    _B00_2 = __zNorth_3DEC_block(ii,jj)
    _x_surf = _B00_1
    _z_surf = _B00_2
    __table_num = int(__ntable_INTERFACE)
    __interpolate_surface ;      %% call interpolation function...
    _y_surf = __y_surf
    _B00_3_TOP = __y_surf
    if _flag_hit_outsider = 1
      _flag_outsider_TRIAG_1 = 1
    end if

    _B10_1 = __xEast_3DEC_block(ii+1,jj)
    _B10_2 = __zNorth_3DEC_block(ii+1,jj)
    _x_surf = _B10_1
    _z_surf = _B10_2
    __interpolate_surface ;      %% call interpolation function...
    _y_surf = __y_surf
    _B10_3_TOP = __y_surf
    if _flag_hit_outsider = 1
      _flag_outsider_TRIAG_2 = 1
    end if

    _B11_1 = __xEast_3DEC_block(ii+1,jj+1)
    _B11_2 = __zNorth_3DEC_block(ii+1,jj+1)
    _x_surf = _B11_1
    _z_surf = _B11_2
    __interpolate_surface ;      %% call interpolation function...
    _y_surf = __y_surf
    _B11_3_TOP = __y_surf
    if _flag_hit_outsider = 1
      _flag_outsider_TRIAG_3 = 1
    end if

    _B01_1 = __xEast_3DEC_block(ii,jj+1)
    _B01_2 = __zNorth_3DEC_block(ii,jj+1)
    _x_surf = _B01_1
    _z_surf = _B01_2
    __interpolate_surface ;      %% call interpolation function...
    _y_surf = __y_surf
    _B01_3_TOP = __y_surf
    if _flag_hit_outsider = 1
      _flag_outsider_TRIAG_4 = 1
    end if

    _B55_1 = 0.25*( _B00_1+ _B10_1+ _B11_1+ _B01_1)
    _B55_2 = 0.25*( _B00_2+ _B10_2+ _B11_2+ _B01_2)
    _x_surf = _B55_1
    _z_surf = _B55_2
_interpolate_surface ; <-- call interpolation function...
_y_surf = _y_surf
_B55_3_TOP = _y_surf
if _flag_hit_outsider = 1
    _flag_outsider_TRIAG_5 = 1
end if
_flag_block_is_insider = 1
if _flag_outsider_TRIAG_1 = 1
    if _flag_outsider_TRIAG_2 = 1
        if _flag_outsider_TRIAG_3 = 1
            if _flag_outsider_TRIAG_4 = 1
                _flag_block_is_insider = 0
            end if
        end if
    end if
end if
end if
; BEGIN CUTTING BLOCKS...
if _flag_block_is_insider = 1 ; <---- BEGIN 'block is an insider...
_max_dip_angle_to_cut = 85.0; <-- maximum angle (in degrees) to cut boundary blocks...
_xmin_BLOCK = 0.5*( _B00_1+_B01_1 )
_xmax_BLOCK = 0.5*( _B10_1+_B11_1 )
_zmin_BLOCK = 0.5*( _B00_2+_B10_2 )
_zmax_BLOCK = 0.5*( _B01_2+_B11_2 )
_dd_bisect_1 = 45.0; <-- do not change!
_dip_bisect_1 = 90.0; <-- do not change!
_dd_bisect_2 = 135.0; <-- do not change!
_dip_bisect_2 = _dip_bisect_1; <-- do not change!
_xP_bisect = _B55_1
_yP_bisect = _B55_3
_zP_bisect = _B55_2
command
    hide
    seek xrange _xmin_BLOCK,_xmax_BLOCK zrange _zmin_BLOCK,_zmax_BLOCK
end_command
;prisms must only be cut on first shear created
if _nregion_above_INTERFACE = 1
    command
        jset origin _xP_bisect,_yP_bisect,_zP_bisect &
        dip=_dip_bisect_1 dd=_dd_bisect_1
        jset origin _xP_bisect,_yP_bisect,_zP_bisect &
        dip=_dip_bisect_2 dd=_dd_bisect_2
    endcommand
end if
; CUT 1st PRISM
command
    hide
    seek xrange _xmin_BLOCK,_xmax_BLOCK zrange _zmin_BLOCK,_zmax_BLOCK
    hide above origin _xP_bisect,_yP_bisect,_zP_bisect &
    dip=_dip_bisect_1 dd=_dd_bisect_1
    hide below origin _xP_bisect,_yP_bisect,_zP_bisect &
    dip=_dip_bisect_2 dd=_dd_bisect_2
endcommand
_pC_1 = _B00_1
_pC_2 = _B00_2
_pC_3 = _B00_3_TOP
_pA_1 = _B10_1

360
function_get_dd_and_dip ; <-- call the function to get the dd and dip of the bottom layer...
_dd_deg = _dd_deg ; <-- returned by 'function_get_dd_and_dip'
_dip_deg = _dip_deg ; <-- returned by 'function_get_dd_and_dip'
if abs(_dip_deg) < _max_dip_angle_to_cut
  command
    jset origin _pC_1, pC_3, pC_2 &
    dip= _dip_deg dd= _dd_deg
    hide below origin _pC_1, pC_3, pC_2 &
    dip= _dip_deg dd= _dd_deg
    mark region _nregion_above_INTERFACE
  end_command
end_if
; CUT 2nd PRISM
command
  hide
  seek xrange _xmin_BLOCK, _xmax_BLOCK zrange _zmin_BLOCK, _zmax_BLOCK
  hide below origin _xP_bisect, _yP_bisect, _zP_bisect &
  dip= _dip_bisect_1 dd= _dd_bisect_1
  hide below origin _xP_bisect, _yP_bisect, _zP_bisect &
  dip= _dip_bisect_2 dd= _dd_bisect_2
end_command
_pC_1 = _B10_1
_pC_2 = _B10_2
_pC_3 = _B10_3_TOP
_pA_1 = _B11_1
_pA_2 = _B11_2
_pA_3 = _B11_3_TOP
_pB_1 = _B55_1
_pB_2 = _B55_2
_pB_3 = _B55_3_TOP
_function_get_dd_and_dip ; <-- call the function to get the dd and dip of the bottom layer...
_dd_deg = _dd_deg ; <-- returned by 'function_get_dd_and_dip'
_dip_deg = _dip_deg ; <-- returned by 'function_get_dd_and_dip'
if abs(_dip_deg) < _max_dip_angle_to_cut
  command
    jset origin _pC_1, pC_3, pC_2 &
    dip= _dip_deg dd= _dd_deg
    hide below origin _pC_1, pC_3, pC_2 &
    dip= _dip_deg dd= _dd_deg
    mark region _nregion_above_INTERFACE
  end_command
end_if
; CUT 3rd PRISM
command
  hide
  seek xrange _xmin_BLOCK, _xmax_BLOCK zrange _zmin_BLOCK, _zmax_BLOCK
  hide below origin _xP_bisect, _yP_bisect, _zP_bisect &
  dip= _dip_bisect_1 dd= _dd_bisect_1
  hide above origin _xP_bisect, _yP_bisect, _zP_bisect &
  dip= _dip_bisect_2 dd= _dd_bisect_2
end_command

361
function get dd and dip ; <-- call the function to get the dd and dip of the bottom layer...
dd_deg = _dd_deg ; <-- returned by '_function_get_dd_and_dip'
dip_deg = _dip_deg ; <-- returned by '_function_get_dd_and_dip'
if abs(dip_deg) < _max_dip_angle_to_cut
command
  jset origin _pC_1, _pC_3, _pC_2 &
  dip= dip_deg dd= dd_deg
  hide below origin _pC_1, _pC_3, _pC_2 &
  dip= dip_deg dd= dd_deg
  mark region _nregion_above_INTERFACE
end_command
end_if
end_if ; <---- END 'block is an insider...' 
; END CUTTING BLOCKS...
end_loop
end_loop
END
B.2.4 Generate Continuum Mesh

This file discretizes continuum materials into mesh elements.

**stage3_mesh_generator.dat**

;mark small blocks at slope boundaries to avoid unrealistic mesh stretching due to
tensile failure at slide boundaries
hide
seek volume 10000
hide region 0
mark region 99
hide
seek volume 20000
hide region 0
hide region 99
mark region 98
seek

DEF _variables
    _char_len_zones_slide = 100.0/4
    _char_len_zones_insitu = 100.0
END
_variables

hide region 0
gen edge _char_len_zones_slide
seek
gen edge _char_len_zones_insitu

save stage3.sav
return
B.2.5 Set-up Instruments

This file sets up "instruments" for tracking deformation at the same relative geographic location as actual monuments are located in the field.

```
stage4_load_instruments.dat

DEF _determine_number_zones
  _counter = 0
  _iBlock = block_head
loop while _iBlock # 0
  _iZone = b_zone(_iBlock)
  loop while _iZone # 0
    _counter = _counter + 1
    _iZone = z_next(_iZone)
  end_loop
  _iBlock = b_next(_iBlock)
end_loop
_number_zones = _counter
END
_determine_number_zones

DEF _fill_arrays_pointers
array _z_extra(_number_zones,3)
  _counter = 0
  _iBlock = block_head
loop while _iBlock # 0
  _iZone = b_zone(_iBlock)
  loop while _iZone # 0
    _counter = _counter + 1
    _z_extra(_counter,1) = _iZone
    _iZone = z_next(_iZone)
  end_loop
  _iBlock = b_next(_iBlock)
end_loop
END
_fill_arrays_pointers

DEF _dot_product  ; to be used later when computing tributary areas for gridpoints
  _mag_d = _ax*_bx + _ay*_by + _az*_bz
END

DEF _cross_product  ; to be used later when computing tributary areas for gridpoints
  _cx = _ay*_bz - _az*_by
  _cy = _az*_bx - _ax*_bz
  _cz = _ax*_by - _ay*_bx
  _mag_c = sqrt(_cx*_cx + _cy*_cy + _cz*_cz)
END

DEF _compute_char_area
  loop i (1,_number_zones)
    _pnt = _z_extra(i,1)
    _pnt_gp1 = z_gp(_pnt,1)
    _pnt_gp2 = z_gp(_pnt,2)
```
_pnt_gp3 = z_gp(_pnt,3)
_pnt_gp4 = z_gp(_pnt,4)
_x_gp1 = gp_x(_pnt_gp1)
_y_gp1 = gp_y(_pnt_gp1)
_z_gp1 = gp_z(_pnt_gp1)
_x_gp2 = gp_x(_pnt_gp2)
_y_gp2 = gp_y(_pnt_gp2)
_z_gp2 = gp_z(_pnt_gp2)
_x_gp3 = gp_x(_pnt_gp3)
_y_gp3 = gp_y(_pnt_gp3)
_z_gp3 = gp_z(_pnt_gp3)
_x_gp4 = gp_x(_pnt_gp4)
_y_gp4 = gp_y(_pnt_gp4)
_z_gp4 = gp_z(_pnt_gp4)
_ax = _x_gp2 - _x_gp1
_ay = _y_gp2 - _y_gp1
_az = _z_gp2 - _z_gp1
_bx = _x_gp3 - _x_gp1
_by = _y_gp3 - _y_gp1
_bz = _z_gp3 - _z_gp1
_cross_product
_area1 = 0.5*abs(_mag_c)
_ax = _x_gp2 - _x_gp1
_ay = _y_gp2 - _y_gp1
_az = _z_gp2 - _z_gp1
_bx = _x_gp4 - _x_gp1
_by = _y_gp4 - _y_gp1
_bz = _z_gp4 - _z_gp1
_cross_product
_area2 = 0.5*abs(_mag_c)
_ax = _x_gp3 - _x_gp1
_ay = _y_gp3 - _y_gp1
_az = _z_gp3 - _z_gp1
_bx = _x_gp4 - _x_gp1
_by = _y_gp4 - _y_gp1
_bz = _z_gp4 - _z_gp1
_cross_product
_area3 = 0.5*abs(_mag_c)
_ax = _x_gp3 - _x_gp2
_ay = _y_gp3 - _y_gp2
_az = _z_gp3 - _z_gp2
_bx = _x_gp4 - _x_gp2
_by = _y_gp4 - _y_gp2
_bz = _z_gp4 - _z_gp2
_cross_product
_area4 = 0.5*abs(_mag_c)
_area = (_area1+_area2+_area3+_area4) / 4.0
_z_extra(i,2) = _area
end_loop
END
_compute_char_area

DEF _variables_instrumentation
array_info_instr(35,8);<---_info_instr(a,b)
;‘a’ is the id of the instrument
;‘b=1’ x-east coordinate of the instrument collar
; 'b=2' z-north coordinate of the instrument collar
; 'b=3' length instrument
; 'b=4' is the dd (dip-direction) in degrees of the instrument (from North, + clockwise)
; 'b=5' is the dip (dip-angle) in degrees of the instrument (from horizontal, + downwards)
; 'b=6' number of points in instrument
; 'b=7' y-upward coordinate of the instrument collar --- computed based on topography...

array_data_instr(35,40,7); <-- data_instr(c,d,e)

;'c' is the id of the instrument
;'d' is the id of the gridpoint in the instrument
;'e=1' is the x-eastern coordinate of the gridpoint in the instrument
;'e=2' is the z-northern coordinate of the gridpoint in the instrument
;'e=3' is the y-upward coordinate of the gridpoint in the instrument
;'e=4' is the id of the zone used to interpolate displacements
;'e=5' is table id with information time vs x-displ
;'e=6' is table id with information time vs y-displ
;'e=7' is table id with information time vs z-displ

_num_instr = 35 ; note more instruments are built than those compared to field data

;S33
_info_instr(1,1) = 1126.0
_info_instr(1,2) = 2077.0
_info_instr(1,3) = 140.0
_info_instr(1,4) = 0.0
_info_instr(1,5) = 90.0
_info_instr(1,6) = 40
_info_instr(1,8) = 23

;M42
_info_instr(2,1) = 2014.4
_info_instr(2,2) = 1199.5
_info_instr(2,3) = 240.0
_info_instr(2,4) = 0.0
_info_instr(2,5) = 90.0
_info_instr(2,6) = 40
_info_instr(2,8) = 10

;S12/M52
_info_instr(3,1) = 3139.0
_info_instr(3,2) = 1737.0
_info_instr(3,3) = 300.0
_info_instr(3,4) = 0.0
_info_instr(3,5) = 90.0
_info_instr(3,6) = 40
_info_instr(3,8) = 30

;M56
_info_instr(4,1) = 3155.1
_info_instr(4,2) = 3262.6
_info_instr(4,3) = 300.0
_info_instr(4,4) = 0.0
_info_instr(4,5) = 90.0
_infoInstr(4,6) = 40
_info_instr(4,8) = 19

;S43
_info_instr(5,1) = 3295.0
_info_instr(5,2) = 3078.0
_info_instr(5,3) = 200.0
_info_instr(5,4) = 0.0
_info_instr(5,5) = 90.0
_info_instr(5,6) = 40

366
<table>
<thead>
<tr>
<th></th>
<th>value 1</th>
<th>value 2</th>
<th>value 3</th>
<th>value 4</th>
<th>value 5</th>
<th>value 6</th>
<th>value 8</th>
</tr>
</thead>
<tbody>
<tr>
<td>S02</td>
<td>3690.0</td>
<td>2048.0</td>
<td>200.0</td>
<td>0.0</td>
<td>90.0</td>
<td>40</td>
<td>35</td>
</tr>
<tr>
<td>S07</td>
<td>2576.99</td>
<td>2051.26</td>
<td>280.0</td>
<td>0.0</td>
<td>90.0</td>
<td>40</td>
<td>33</td>
</tr>
<tr>
<td>S09</td>
<td>2908.0</td>
<td>2252.0</td>
<td>300.0</td>
<td>0.0</td>
<td>90.0</td>
<td>40</td>
<td>32</td>
</tr>
<tr>
<td>S14</td>
<td>3446.0</td>
<td>2605.0</td>
<td>240.0</td>
<td>0.0</td>
<td>90.0</td>
<td>40</td>
<td>29</td>
</tr>
<tr>
<td>S23</td>
<td>1950.0</td>
<td>2645.0</td>
<td>160.0</td>
<td>0.0</td>
<td>90.0</td>
<td>40</td>
<td>32</td>
</tr>
<tr>
<td>Instruction</td>
<td>Memory 1</td>
<td>Memory 2</td>
<td>Memory 3</td>
<td>Memory 4</td>
<td>Memory 5</td>
<td>Memory 6</td>
<td>Memory 8</td>
</tr>
<tr>
<td>-------------</td>
<td>----------</td>
<td>----------</td>
<td>----------</td>
<td>----------</td>
<td>----------</td>
<td>----------</td>
<td>----------</td>
</tr>
<tr>
<td>12</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>4146.0</td>
<td>1636.0</td>
<td>100.0</td>
<td>0.0</td>
<td>90.0</td>
<td>40</td>
<td>36</td>
</tr>
<tr>
<td>14</td>
<td>3563.0</td>
<td>2612.0</td>
<td>160.0</td>
<td>0.0</td>
<td>90.0</td>
<td>40</td>
<td>30</td>
</tr>
<tr>
<td>15</td>
<td>2747.0</td>
<td>3045.0</td>
<td>300.0</td>
<td>0.0</td>
<td>90.0</td>
<td>40</td>
<td>31</td>
</tr>
<tr>
<td>16</td>
<td>1912.0</td>
<td>1619.0</td>
<td>240.0</td>
<td>0.0</td>
<td>90.0</td>
<td>40</td>
<td>30</td>
</tr>
<tr>
<td>17</td>
<td>1553.8</td>
<td>1688.0</td>
<td>200.0</td>
<td>0.0</td>
<td>90.0</td>
<td>40</td>
<td>12</td>
</tr>
<tr>
<td>18</td>
<td>2024.6</td>
<td>2092.6</td>
<td>260.0</td>
<td>0.0</td>
<td>90.0</td>
<td>40</td>
<td>26</td>
</tr>
<tr>
<td>19</td>
<td>1363.1</td>
<td>2083.0</td>
<td>240.0</td>
<td>0.0</td>
<td>90.0</td>
<td>40</td>
<td>40</td>
</tr>
</tbody>
</table>
DEF _is_triangle_insider
  _flag_negative_dot_product = 0
  ; Check side P1-P2
  ; get normal vector to side first...
  _ax = _xP2 - _xP1
  _ay = _yP2 - _yP1
  _az = 1.0
  _bx = _xP2 - _xP1
  _by = _yP2 - _yP1
  _bz = 0.0
  _cross_product
  _cx = _cx
  _cy = _cy
  _cz = _cz

  ; get dot product of normal and vector to the point...
  _ax = _xP - _xP1
  _ay = _yP - _yP1
  _az = 0.0
  _bx = _cx
  _by = _cy
  _bz = _cz
  _dot_product
  _mag_d = _mag_d
  if _mag_d < 0.0
    _flag_negative_dot_product = 1
  end_if

  ; Check side P2-P3
  ; get normal vector to side first...
  _ax = _xP3 - _xP2
  _ay = _yP3 - _yP2
  _az = 1.0
  _bx = _xP3 - _xP2
  _by = _yP3 - _yP2
  _bz = 0.0
  _cross_product
  _cx = _cx
; get dot product of normal and vector to the point...
_gx = _xP - _xP2
_gy = _yP - _yP2
_gz = 0.0
_gb = _cx
_gb = _cy
_gb = _cz
_dot_product
_mag_d = _mag_d
if _mag_d < 0.0
  _flag_negative_dot_product = 1
end_if
; Check side P3-P1
; get normal vector to side first...
_gx = _xP1 - _xP3
_gy = _yP1 - _yP3
_gz = 1.0
_gb = _xP1 - _xP3
_gb = _yP1 - _yP3
_gb = 0.0
_cross_product
_gx = _cx
_gb = _cy
_gb = _cz
; get dot product of normal and vector to the point...
_gx = _xP - _xP3
_gb = _yP - _yP3
_gz = 0.0
_gb = _cx
_gb = _cy
_gb = _cz
_dot_product
_mag_d = _mag_d
if _mag_d < 0.0
  _flag_negative_dot_product = 1
end_if
if _flag_negative_dot_product = 0
  _is_triangle_insider = 1
else
  _is_triangle_insider = 0
end_if
END

DEF _interpolate_triangle
_t1=_uk*_xj*_yi*_uj*_xk*_yi*_yi*_uk*_xi*_yj*_ui*_xk*_yi*_ui*_xj*_yk;
_t2=-_xj*_yi+_xk*_yi+_xi*_yj-_xk*_yj-_xi*_yk+_xj*_yk;
_a1=-_t1/_t2
_t1=_uj*_yi*_yi*_uk*_yi+_uk*_yi+_ui*_yk+-_uk*_xj*_yk;
_t2=-_xj*_yi+_xk*_yi+_xi*_yj-_xk*_yj-_xi*_yk+_xj*_yk;
_a2=-_t1/_t2
_t1=-_uj*_xi+_uk*_xi+_ui*_xj*_yj-_ui*_xj*_yk;
_t2=-_xj*_yi+_xk*_yi+_xi*_yj-_xk*_yj-_xi*_yk+_xj*_yk;
_a3=-_t1/_t2
_u_point = _a1+_a2_*_x+_a3_*_y
DEF _compute_elevation collar instruments
loop instr (1, num instr)
junk = out(string( num instr))
_flag_found_interval = 0
_xeast_INSTR = _info_instr( instr, 1)
_znorth_INSTR = _infoinstr( _instr, 2)
loop ii (1, npts xEast)
    loop jj (1, npts zNorth)
        _xeast_MIN_ = _xEast_3DEC_block(ii, jj)
        _xeast_MAX_ = _xEast_3DEC_block(ii+1, jj)
        _znorth_MIN_ = _zNorth_3DEC_block(ii, jj)
        _znorth_MAX_ = _zNorth_3DEC_block(ii, jj+1)
        if _flag_found_interval = 0
            if _xeast_INSTR <= _xeast_MAX_
                if _xeast_INSTR >= _xeast_MIN_
                    if _znorth_INSTR <= _znorth_MAX_
                        if _znorth_INSTR >= _znorth_MIN_
                            _ii_BASE = ii
                            _jj_BASE = jj
                            _flag_found_interval = 1
                        end if
                    end if
                end if
            end if
        end if
end loop
end loop
if _flag_found_interval = 0
    _dummy = out('could not find interpolation interval...')
    command
    stop
end if
; ----- analyze in which of the 4 triangles the point falls into...
_flag_falls_triangle_1 = 0
_flag_falls_triangle_2 = 0
_flag_falls_triangle_3 = 0
_flag_falls_triangle_4 = 0
_x00 = _xEast_3DEC_block( _ii_BASE, _jj_BASE ) ; <-- x and y are planar cartesian coordinates...
_y00 = _zNorth_3DEC_block( _ii_BASE, _jj_BASE )
_z00 = _yUpward_3DEC_block( _ii_BASE, _jj_BASE )
_x10 = _xEast_3DEC_block( _ii_BASE+1, _jj_BASE )
_y10 = _zNorth_3DEC_block( _ii_BASE+1, _jj_BASE )
_z10 = _yUpward_3DEC_block( _ii_BASE+1, _jj_BASE )
_x11 = _xEast_3DEC_block( _ii_BASE+1, _jj_BASE+1 )
_y11 = _zNorth_3DEC_block( _ii_BASE+1, _jj_BASE+1 )
_z11 = _yUpward_3DEC_block( _ii_BASE+1, _jj_BASE+1 )
_x01 = _xEast_3DEC_block( _ii_BASE, _jj_BASE+1 )
_y01 = _zNorth_3DEC_block( _ii_BASE, _jj_BASE+1 )
_z01 = _yUpward_3DEC_block( _ii_BASE, _jj_BASE+1 )
_xP = _xeast_INSTR
_yP = _znorth_INSTR
; analyze triangle #1
_xP1 = _x00
if _status = 1
    _falls_triangle_1 = 1
end_if

; analyze triangle #2
_xP1 = _x11
_yP1 = _y11
_xP2 = _x01
_yP2 = _y01
_xP3 = _x00
_yP3 = _y00
_status = _is_triangle_insider
if _status = 1
    _falls_triangle_2 = 1
end_if

; analyze triangle #3
_xP1 = _x00
_yP1 = _y00
_xP2 = _x10
_yP2 = _y10
_xP3 = _x11
_yP3 = _y11
_status = _is_triangle_insider
if _status = 1
    _falls_triangle_3 = 1
end_if

; analyze triangle #4
_xP1 = _x01
_yP1 = _y01
_xP2 = _x10
_yP2 = _y10
_xP3 = _x11
_yP3 = _y11
_status = _is_triangle_insider
if _status = 1
    _falls_triangle_4 = 1
end_if

; ---- Perform interpolation triangles...
_triangles_computed = 0
_elevation = 0.0
if _falls_triangle_1 = 1
    _xi=_x00
    _yi=_y00
    _ui=_z00
    _xj=_x10
    _yj=_y10
    _uj=_z10
    _xk=_x11
    _yk=_y11
    _uk=_z11
    _x=_xP
y = yP
_interpolate_triangle
_elevation = _elevation + _u point
_triangles_computed = _triangles_computed + 1
_dummy = out(Triangle #1 -- _u point: ' + string(_u point))
end if
if _falls_triangle_2 = 1
   _xi = _x11
   _yi = _y11
   _ui = _z11
   _xj = _x01
   _yj = _y01
   _uj = _z01
   _xk = _x00
   _yk = _y00
   _uk = _z00
   _x = _xP
   _y = yP
   _interpolate_triangle
   _elevation = _elevation + _u point
   _triangles_computed = _triangles_computed + 1
   _dummy = out(Triangle #2 -- _u point: ' + string(_u point))
end if
if _triangles_computed < 2
   if _falls_triangle_3 = 1
      _xi = _x00
      _yi = _y00
      _ui = _z00
      _xj = _x10
      _yj = _y10
      _uj = _z10
      _xk = _x01
      _yk = _y01
      _uk = _z01
      _x = _xP
      _y = yP
      _interpolate_triangle
      _elevation = _elevation + _u point
      _triangles_computed = _triangles_computed + 1
      _dummy = out(Triangle #3 -- _u point: ' + string(_u point))
   end if
end if
if _triangles_computed < 2
   if _falls_triangle_4 = 1
      _xi = _x01
      _yi = _y01
      _ui = _z01
      _xj = _x10
      _yj = _y10
      _uj = _z10
      _xk = _x11
      _yk = _y11
      _uk = _z11
      _x = _xP
      _y = yP
      _interpolate_triangle
_elevation = _elevation + _u_point
_triangles_computed = _triangles_computed + 1
 dummy = out('Triangle #4 --> _u_point: ' + string(_u_point))
end if
end if
if _triangles_computed # 2
 dummy = out('error in locating triangles: _triangles_computed # 2 !!! ')
command
pause
end_command
end if
_elevation = 0.5 * _elevation ; <--- compute the elevation of the point...
SHIFT_elev_collar = 10.0 ; <--- used to move the collar downwards so it lies below the surface...
info_instr(_instr,7) = _elevation - SHIFT_elev_collar
end_loop
END
_compute_elevation_collar_instruments

DEF _compute_coords_points_instruments
loop k (1, _num_instr)
 Len_instr = info_instr(k, 3)
 dd = info_instr(k, 4)
 dip = info_instr(k, 5)
 npts = info_instr(k, 6)
 xeast_collar = info_instr(k, 1)
 znorth_collar = info_instr(k, 2)
 yupward_collar = info_instr(k, 7)
loop i (1, npts)
 Len = Len_instr * float(i-1)/float(npts-1)
 del_yupward = -Len*sin(dip*pi/180.0)
 del_znorth = Len*cos(dip*pi/180.0)*sin(dd*pi/180.0)
 xeast_pt = xeast_collar + del_xeast
 znorth_pt = znorth_collar + del_znorth
 yupward_pt = yupward_collar + del_yupward
 data_instr(k,i,1) = xeast_pt
 data_instr(k,i,2) = znorth_pt
 data_instr(k,i,3) = yupward_pt
end_loop
end_loop
END
_compute_coords_points_instruments

DEF _define_storage_tables_for_points_instruments
_table_id = int(_ntable_INSTRUMENTATION - 1)
loop k (1, _num_instr)
 npts = info_instr(k, 6)
loop i (1, npts)
 _table_id = int(_table_id + 1)
 data_instr(k,i,5) = int(_table_id)
 _table_id = int(_table_id + 1)
 data_instr(k,i,6) = int(_table_id)
 _table_id = int(_table_id + 1)
 data_instr(k,i,7) = int(_table_id)
end_loop
end_loop
; Locate zones where instrument gridpoints are falling...

DEF _store_max_radius_to_vertex_in_zones
loop k (1, _number_zones)
  _pnt = _z_extra(k,1)
  _z_xcen = z_x(_pnt)
  _z_ycen = z_y(_pnt)
  _z_zcen = z_z(_pnt)
  _max_radius = -1e10
  loop i(1,4)
    _pnt_gp = z_gp(_pnt,i)
    _gp_x = gp_x(_pnt_gp)
    _gp_y = gp_y(_pnt_gp)
    _gp_z = gp_z(_pnt_gp)
    _rad = sqrt((_gp_x-_z_xcen)^2+_gp_y-_z_ycen)^2+(_gp_z-_z_zcen)^2)
    if _rad > _max_radius
      _max_radius = _rad
    end_if
  end_loop
  _z_extra(k,3) = _max_radius
end_loop
END

DEF _is_tetrahedra_insider
_loop = _zone_pnt
_pnt_gp1 = z_gp(_pnt,1)
_pnt_gp2 = z_gp(_pnt,2)
_pnt_gp3 = z_gp(_pnt,3)
_pnt_gp4 = z_gp(_pnt,4)
_x_gp1 = gp_x(_pnt_gp1)
_y_gp1 = gp_z(_pnt_gp1)
_z_gp1 = gp_z(_pnt_gp1)
_x_gp2 = gp_x(_pnt_gp1)
_y_gp2 = gp_y(_pnt_gp1)
_z_gp2 = gp_z(_pnt_gp1)
_x_gp3 = gp_x(_pnt_gp1)
_y_gp3 = gp_y(_pnt_gp1)
_z_gp3 = gp_z(_pnt_gp1)
_x_gp4 = gp_x(_pnt_gp1)
_y_gp4 = gp_y(_pnt_gp1)
_z_gp4 = gp_z(_pnt_gp1)
_flag_sign_all_positive = 1
; --- BEGIN: Face 123
ax = _x_gp2 - _x_gp1
ay = _z_gp2 - _z_gp1
az = _y_gp2 - _y_gp1
bx = _x_gp3 - _x_gp1
by = _z_gp3 - _z_gp1
bz = _y_gp3 - _y_gp1
_cross_product ; <-- perform the cross product...
 cx = _cx
cy = _cy
cz = _cz
\_ax = \_x - \_x\_gp1 \\
\_ay = \_z - \_z\_gp1 \\
\_az = \_y - \_y\_gp1 \\
\_bx = \_cx \\
\_by = \_cy \\
\_bz = \_cz \\
_\text{dot\_product} \\
_\text{mag\_d} = _\text{mag\_d} \\
_\text{dummy} = \text{out('Face 123 -> _mag\_d: ' + string(_mag\_d))} \\
\text{if } _\text{mag\_d} < 0.0 \\
\quad _\text{flag\_sign\_all\_positive} = 0 \\
\text{end\_if} \\
; --- BEGIN: Face 124 \\
_\text{ax} = _\text{x\_gp2} - _\text{x\_gp4} \\
_\text{ay} = _\text{z\_gp2} - _\text{z\_gp4} \\
_\text{az} = _\text{y\_gp2} - _\text{y\_gp4} \\
_\text{bx} = _\text{x\_gp1} - _\text{x\_gp4} \\
_\text{by} = _\text{z\_gp1} - _\text{z\_gp4} \\
_\text{bz} = _\text{y\_gp1} - _\text{y\_gp4} \\
_\text{cross\_product} ; \quad \text{<-- perform the cross product...} \\
_\text{cx} = _\text{cx} \\
_\text{cy} = _\text{cy} \\
_\text{cz} = _\text{cz} \\
_\text{ax} = _\text{x} - _\text{x\_gp4} \\
_\text{ay} = _\text{z} - _\text{z\_gp4} \\
_\text{az} = _\text{y} - _\text{y\_gp4} \\
_\text{bx} = _\text{cx} \\
_\text{by} = _\text{cy} \\
_\text{bz} = _\text{cz} \\
_\text{dot\_product} \\
_\text{mag\_d} = _\text{mag\_d} \\
_\text{dummy} = \text{out('Face 124 -> _mag\_d: ' + string(_mag\_d))} \\
\text{if } _\text{mag\_d} < 0.0 \\
\quad _\text{flag\_sign\_all\_positive} = 0 \\
\text{end\_if} \\
; --- BEGIN: Face 234 \\
_\text{ax} = _\text{x\_gp3} - _\text{x\_gp4} \\
_\text{ay} = _\text{z\_gp3} - _\text{z\_gp4} \\
_\text{az} = _\text{y\_gp3} - _\text{y\_gp4} \\
_\text{bx} = _\text{x\_gp2} - _\text{x\_gp4} \\
_\text{by} = _\text{z\_gp2} - _\text{z\_gp4} \\
_\text{bz} = _\text{y\_gp2} - _\text{y\_gp4} \\
_\text{cross\_product} ; \quad \text{<-- perform the cross product...} \\
_\text{cx} = _\text{cx} \\
_\text{cy} = _\text{cy} \\
_\text{cz} = _\text{cz} \\
_\text{ax} = _\text{x} - _\text{x\_gp4} \\
_\text{ay} = _\text{z} - _\text{z\_gp4} \\
_\text{az} = _\text{y} - _\text{y\_gp4} \\
_\text{bx} = _\text{cx} \\
_\text{by} = _\text{cy} \\
_\text{bz} = _\text{cz} \\
_\text{dot\_product} \\
_\text{mag\_d} = _\text{mag\_d} \\
_\text{dummy} = \text{out('Face 234 -> _mag\_d: ' + string(_mag\_d))} \\
\text{if } _\text{mag\_d} < 0.0 

FLAG_SIGN_ALL_POSITIVE = 0

END_IF

; --- BEGIN: Face 134

AX = X_GP1 - X_GP4

AY = Z_GP1 - Z_GP4

AZ = Y_GP1 - Y_GP4

BX = X_GP3 - X_GP4

BY = Z_GP3 - Z_GP4

BZ = Y_GP3 - Y_GP4

CROSS_PRODUCT; <-- perform the cross product...

CX = _cx

CY = _cy

CZ = _cz

AX = _x - _x_GP4

AY = _z - _z_GP4

AZ = _y - _y_GP4

BX = _cx

BY = _cy

BZ = _cz

DOT_PRODUCT

MAG_D = MAG_D

DUMMY = OUT('Face 134 -> MAG_D: ' + STRING(MAG_D))

IF MAG_D < 0.0

FLAG_SIGN_ALL_POSITIVE = 0

ENDIF

FLAG_IT_IS_INSIDER = 0

IF FLAG_SIGN_ALL_POSITIVE = 1

FLAG_IT_IS_INSIDER = 1

ENDIF

DUMMY = OUT('---------------------')

END

DEF DEF_interp_zones_for_instr_gpts

NTAB_TEMP = INT(10); <-- id of the table where temporary information will be stored...

LOOP _instr (1, NUM_INSTR)

NPTS = INFO_INSTR(_instr, 6)

LOOP GPT_INSTR (1, NPTS); <-- analyze the different instrumentation gpts...

GP_X = DATA_INSTR(_instr, GPT_INSTR, 1)

GP_Y = DATA_INSTR(_instr, GPT_INSTR, 3)

GP_Z = DATA_INSTR(_instr, GPT_INSTR, 2)

LOOP THROUGH ALL ZONES AND DETECT THE CANDIDATE ZONES TO CONTAIN THE INSTRUMENT GPT...

COUNTER = 0

LOOP K (1, NUMBER_ZONES)

PNT = Z_EXTRA(K, 1)

Z_XCEN = Z_X(PNT)

Z_YCEN = Z_Y(PNT)

Z_ZCEN = Z_Z(PNT)

MAX_RAD = Z_EXTRA(K, 3)

RAD_INSTR = SQRT((GP_X - Z_XCEN)^2 + (GP_Y - Z_YCEN)^2 + (GP_Z - Z_ZCEN)^2)

IF RAD_INSTR <= MAX_RAD

COUNTER = COUNTER + 1

XTABLE(NTAB_TEMP, COUNTER) = COUNTER

YTABLE(NTAB_TEMP, COUNTER) = K

END_IF

END_LOOP

IF COUNTER = 0

379

END_INTERP_ZONES_FOR_INSTR_GPTS
; loop through candidate zones and detect the actual interpolation zone...

_npts_table = table_size(_ntab_temp)
_flag_found_and_insider = 0
loop j (1, _npts_table)
  k = int(ytable(_ntab_temp,j))
  _zone_pnt = _z_extra(k,1)
  _x = _gp_x
  _y = _gp_y
  _z = _gp_y
  _is_tetrahedra_insider
  if _flag_found_and_insider = 0
    if _flag_it_is_insider = 1
      _dummy = out('DETECTED INSIDER...')
      _flag_found_and_insider = 1
    ; store the id of the zone used to interpolate displacements in the info matrix...
    _data_instr(_instr, _gpt_instr, 4) = _zone_pnt
  end_if
end_if
end_loop
if _flag_found_and_insider = 0
  _line = 'error (2) in function _def_interp_zones_for_instr_gpts!!!!'
  _dummy = out(_line)
  command
  pause
end_command
end_if
; END: loop through candidate zones and detect the actual interpolation zone..
command
  table _ntab_temp erase
end_command
end_loop
END

_DEF_interpolate_tetrahedra

_t = _up*_xm*_yj*_zi-_um*_xp*_yj*_zi-_up*_xj*_ym*_zi+_uj*_xp*_ym*_zi
_t = _t + _um*_xj*_yp*_zi-_uj*_xm*_yp*_zi-_up*_xm*_yi*_zj+
_t = _t + _up*_xi*_ym*_zj-_ui*_xp*_ym*_zj-_um*_xi*_yp*_zj
_t = _t + _up*_xj*_yi*_zm-_uj*_xp*_yi*_zm-_up*_xi*_yj*_zm+
_t = _t + _uj*_xi*_yp*_zm-_ui*_xj*_yp*_zm-_um*_xj*_yi*_zp+
_t = _t + _um*_xi*_yj*_zp-_ui*_xm*_yj*_zp-_uj*_xi*_ym*_zp+
_num1 = _t
_t = _xm*_yj*_zi-_xp*_yj*_zi-_xj*_ym*_zi+_xp*_ym*_zi+_xj*_yp*_zi-
_t = _t + _xm*_yi*_zj+_xp*_yi*_zj+_ji*_yi*_yi*_zj+
_t = _t + _xp*_yi*_ym*_zj-_xp*_yi*_ym*_zj-_xp*_yi*_ym*_zj+
_t = _t + _xm*_yp*_zj+_xj*_yi*_zm+_xp*_yi*_zm+

380
\[ t = t + x_i \cdot y_j \cdot z_m - x_j \cdot y_i \cdot z_p + x_m \cdot y_i \cdot z_p + x_i \cdot y_j \cdot z_p \]
\[ t = t + x_m \cdot y_j \cdot z_p - x_i \cdot y_m \cdot z_p + x_j \cdot y_m \cdot z_p; \]
\[ \text{den}_1 = t \]
\[ a_1 = \frac{\text{num}_1}{\text{den}_1} \]
\[ t = t - x_m \cdot y_j \cdot z_p - x_i \cdot y_m \cdot z_p + x_j \cdot y_m \cdot z_p; \]
\[ \text{num}_2 = t \]
\[ \text{den}_2 = t \]
\[ a_2 = \frac{\text{num}_2}{\text{den}_2} \]
\[ t = t + x_i \cdot y_j \cdot z_i + x_j \cdot y_m \cdot z_i - x_m \cdot y_j \cdot z_i \]
\[ t = t + x_i \cdot y_j \cdot z_p - x_i \cdot y_j \cdot z_p + x_j \cdot y_m \cdot z_p - x_j \cdot y_m \cdot z_p; \]
\[ t = t - x_i \cdot y_j \cdot z_i + x_j \cdot y_m \cdot z_i + x_m \cdot y_j \cdot z_i; \]
\[ \text{num}_3 = t \]
\[ \text{den}_3 = t \]
\[ a_3 = \frac{\text{num}_3}{\text{den}_3} \]
\[ t_1 = ((ui - up) - (ui + up)) \]
\[ t_2 = (xi + yi) + x_m \cdot y_i \cdot z_m + x_i \cdot y_m \cdot z_m \]
\[ t = t_1 \cdot t_2 \]
\[ t = t + t_1 \cdot t_2 \]
\[ \text{num}_4 = t \]
\[ \text{den}_4 = t \]
\[ \text{flag}_\text{singular} = 0 \]
\[ \text{if} \ \text{num}_4 = 0 \]
\[ \text{if} \ \text{den}_4 = 0 \]
\[ \text{flag}_\text{singular} = 1 \]
\[ \text{end if} \]
\[ \text{end if} \]
\[ \text{if} \ \text{flag}_\text{singular} = 1 \]
\[ a_4 = 0.0 \]
\[ \text{else} \]
\[ a_4 = \frac{\text{num}_4}{\text{den}_4} \]
\[ \text{end if} \]
\[ u_\text{point} = a_1 + a_2 \cdot x + a_3 \cdot y + a_4 \cdot z \]

381
DEF _store_displacement_instrumentation
  _flag_first_entry = _flag_first_entry ; <--- it will be zero the first time it enters...
loop kk (1, _num_instr)
  _npts = _info_instr(kk,6)
loop ii (1, _npts)
  _pnt_zone_interp = _data_instr(kk,ii,4)
  _id_table_xdisp = int(_data_instr(kk,ii,5))
  _id_table_ydisp = int(_data_instr(kk,ii,6))
  _id_table_zdisp = int(_data_instr(kk,ii,7))
  ; --- get current accumulated displacements...
  if _flag_first_entry # 1.0 ; <--- if this is the first entry...
    _curr_x_disp = 0.0
  else
    _tab_pos = table_size(_id_table_xdisp)
    _curr_x_disp = ytable(_id_table_xdisp, _tab_pos)
  end_if
  if _flag_first_entry # 1.0 ; <--- if this is the first entry...
    _curr_y_disp = 0.0
  else
    _tab_pos = table_size(_id_table_ydisp)
    _curr_y_disp = ytable(_id_table_ydisp, _tab_pos)
  end_if
  if _flag_first_entry # 1.0 ; <--- if this is the first entry...
    _curr_z_disp = 0.0
  else
    _tab_pos = table_size(_id_table_zdisp)
    _curr_z_disp = ytable(_id_table_zdisp, _tab_pos)
  end_if
  ;-------------------------------------------------------------------
  ; Note: as per Mark's suggestion, Carlos added 'large-strain'
  ;       mode for the instrumentation (we add the accumulated displacements
  ;       to the coordinates _gp_x, _gp_y, _gp_z before interpolation...
  ;-------------------------------------------------------------------
  _x = _data_instr(kk,ii,1) + _curr_x_disp
  _y = _data_instr(kk,ii,3) + _curr_y_disp
  _z = _data_instr(kk,ii,2) + _curr_z_disp
  _pnt_gp1 = z_gp(_pnt_zone_interp,1)
  _pnt_gp2 = z_gp(_pnt_zone_interp,2)
  _pnt_gp3 = z_gp(_pnt_zone_interp,3)
  _pnt_gp4 = z_gp(_pnt_zone_interp,4)
  _xi = gp_x(_pnt_gp1)
  _yi = gp_y(_pnt_gp1)
  _zi = gp_z(_pnt_gp1)
  _xj = gp_x(_pnt_gp2)
  _yj = gp_y(_pnt_gp2)
  _zj = gp_z(_pnt_gp2)
  _xm = gp_x(_pnt_gp3)
  _ym = gp_y(_pnt_gp3)
  _zm = gp_z(_pnt_gp3)
  _xp = gp_x(_pnt_gp4)
  _yp = gp_y(_pnt_gp4)
  _zp = gp_z(_pnt_gp4)
 ; interpolate x-disp
  _ui = gp_xdisp(_pnt_gp1)
if _flag_first_entry # 1.0 ; <--- if this is the first entry...
    _tab_pos = 1
else
    _tab_pos = table_size(_id_table_xdisp) + 1
end_if
xtable(_id_table_xdisp,_tab_pos) = time
ytable(_id_table_xdisp,_tab_pos) = _displacement
kk_x = _displacement
; Interpolate y-displacement
ui = gp_ydis(_pnt_gp1)
uj = gp_ydis(_pnt_gp2)
um = gp_ydis(_pnt_gp3)
up = gp_ydis(_pnt_gp4)
_interpolate_tethrahedra
_u_point = _u_point
_displacement = _u_point
if _flag_first_entry # 1.0 ; <--- if this is the first entry...
    _tab_pos = 1
else
    _tab_pos = table_size(_id_table_ydisp) + 1
end_if
xtable(_id_table_ydisp,_tab_pos) = time
ytable(_id_table_ydisp,_tab_pos) = _displacement
kk_y = _displacement
; Interpolate z-displacement
ui = gp_zdis(_pnt_gp1)
uj = gp_zdis(_pnt_gp2)
um = gp_zdis(_pnt_gp3)
up = gp_zdis(_pnt_gp4)
_interpolate_tethrahedra
_u_point = _u_point
_displacement = _u_point
if _flag_first_entry # 1.0 ; <--- if this is the first entry...
    _tab_pos = 1
else
    _tab_pos = table_size(_id_table_zdisp) + 1
end_if
xtable(_id_table_zdisp,_tab_pos) = time
ytable(_id_table_zdisp,_tab_pos) = _displacement
end_loop
end_loop
_flag_first_entry = 1.0
END

DEF _export_instrumentation_records
array _bb(10000000)
_root = 'NN'
_fname_INSTR_Locat = _root + '_[Instr_Locat].txt'
_tab = 't'
; Export gridpoint coordinates
The information in the columns below is as follows:

Column #1: instrument number
Column #2: gridpoint number
Column #3: x-coordinate of the gridpoint
Column #4: y-coordinate of the gridpoint
Column #5: z-coordinate of the gridpoint

loop kk (1, _num_instr)
    _npts = _info_instr(kk,6)
    loop ii (1, _npts)
        _id_instr = kk
        _id_gpt = ii
        _x_pos = _data_instr(kk,ii,1)
        _y_pos = _data_instr(kk,ii,3)
        _z_pos = _data_instr(kk,ii,2)
        _line = string(_id_instr) + _tab + string(_id_gpt) + _tab
        _line = _line + string(_x_pos) + _tab + string(_y_pos) + _tab
        _line = _line + string(_z_pos)
        _nlines = _nlines + 1
        _bb(_nlines) = _line
    end_loop
end_loop

; Export gridpoint displacements
loop kk (1, _num_instr)
    _fname_INSTR_Hist = _root + '[Hist_Instr_'+string(kk)+'].txt'
    _npts = _info_instr(kk,6)
    _nlines = 1
    _line = '----------------------------------------------------------'
    _bb(_nlines) = _line
    _nlines = _nlines + 1
    _line = '-- This is the displacement history information for INSTRUMENT # '
    _line = _line + string(kk) + ' --'
    _bb(_nlines) = _line
    _nlines = _nlines + 1
    _line = '----------------------------------------------------------'
    _bb(_nlines) = _line
    _nlines = _nlines + 1
The information in the columns below is as follows:

Column #1: time (seconds in 3DEC)

Column #2: x-displacement

Column #3: y-displacement

Column #4: z-displacement

This is information for gridpoint #1: 

<table>
<thead>
<tr>
<th>Time (seconds)</th>
<th>X-displacement</th>
<th>Y-displacement</th>
<th>Z-displacement</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
</tr>
</tbody>
</table>

END
B.2.6 Set-up Rockmass Pore Pressures

This file sets up functions to load gridpoint pore pressures

stage5_rockmass_pp.dat

DEF _variables_pore_pressure
  _grav = 9.81
  _dens_water = 1000.0;
  _gamma_water = _grav*_dens_water;
  _grav_NEG = -_grav
END
_variables_pore_pressure

; load water table files (these vary for different groundwater runs
call LOWER_PRE-DRAIN.txt
call UPPER_PRE-DRAIN.txt

DEF _determine_number_gpts
  _counter = 0
  _iBlock = block_head
loop while _iBlock # 0
  _iGp = b_gp(_iBlock)
  loop while _iGp # 0 ;<-- loop through all gridpoints in the block...
    _counter = _counter + 1
    _iGp = gp_next(_iGp)
  end_loop
  _iBlock = b_next(_iBlock)
end_loop
_number_gpts = _counter
END
_determine_number_gpts

DEF _fill_pore_pressure_information
  array _gpt_extra(_number_gpts,3)
    ;<-- 1st column contains the pointer to the gridpoint...
    ;<-- 2nd column contains pore-pressure for base condition
    ;<-- 3rd column contains pore-pressure for change in water table
  _counter = 0
  _iBlock = block_head
loop while _iBlock # 0
  _iGp = b_gp(_iBlock)
  loop while _iGp # 0 ;<-- loop through all gridpoints in the block...
    _counter = _counter + 1
    _gpt_extra(_counter,1) = _iGp
    _iGp = gp_next(_iGp)
  end_loop
  _iBlock = b_next(_iBlock)
end_loop
loop iii (1,_number_gpts)
  _line = 'Processing gridpoint k = '+string(iii)+' of a total of'
  _line = _line +string(_number_gpts) + '...
  _dummy = out(_line)
  _pnt = _gpt_extra(iii,1)
B.2.7 Parameter Initialization and Set-up Sequence

This file defines material properties, constitutive models and runs the initial model set-up sequence (Table 3.1).

stage6_set-up_sequence.dat

set jcondf 2
set jmatdf 1

DEF _material_properties
; rock mass properties for staged set-up
; consolidation A
_density_ROCK = 2700.0
_young_CONS = 1000.0e9 ; <-- high value to avoid large disp of gridpoints during consolidation
_poiss_CONS = 0.49
_bulk_CONS = _young_CONS / 3.0 / (1-2*_poiss_CONS)
_shear_CONS = _young_CONS / 2.0 / (1+_poiss_CONS)
; Consolidation B1
_young_ROCK_B = 100.0e9
_poiss_ROCK_B = 0.35
_bulk_ROCK_B = _young_ROCK_B / 3.0 / (1-2* _poiss_ROCK_B)
_shear_ROCK_B = _young_ROCK_B / 2.0 / (1+ _poiss_ROCK_B)

; Consolidation B2
_young_ROCK_B2 = 10.0e9
_poiss_ROCK_B2 = 0.25
_bulk_ROCK_B2 = _young_ROCK_B2 / 3.0 / (1-2* _poiss_ROCK_B2)
_shear_ROCK_B2 = _young_ROCK_B2 / 2.0 / (1+ _poiss_ROCK_B2)

; Consolidation C - gsi = 45, d = 0
_young_ROCK_C = 10.0e9
_poiss_ROCK_C = 0.25
_fric_ROCK_C = 46
_tens_ROCK_C = 100.0e6
_dilat_ROCK_C = 0.0
_bulk_ROCK_C = _young_ROCK_C / 3.0 / (1-2* _poiss_ROCK_C)
_shear_ROCK_C = _young_ROCK_C / 2.0 / (1+ _poiss_ROCK_C)
_cohe_ROCK_C = 1000.0e6 ;<--- high value to avoid plastic deform and large gridpoint disp

; Consolidation D - d = 0.5 gsi = 40 change mat = 2 in slide
_young_ROCK_D = 1.0e9
_poiss_ROCK_D = 0.25
_fric_ROCK_D = 37.7
_tens_ROCK_D = 0.05e6
_cohe_ROCK_D = 1.0e6
_dilat_ROCK_D = 0.0
_bulk_ROCK_D = _young_ROCK_D / 3.0 / (1-2* _poiss_ROCK_D)
_shear_ROCK_D = _young_ROCK_D / 2.0 / (1+ _poiss_ROCK_D)

; Consolidation D2 - D = 0.7
_young_ROCK_D2 = 0.5e9
_poiss_ROCK_D2 = 0.25
_fric_ROCK_D2 = 34
_tens_ROCK_D2 = 0.04e6
_cohe_ROCK_D2 = 1.0e6
_bulk_ROCK_D2 = _young_ROCK_D2 / 3.0 / (1-2* _poiss_ROCK_D2)
_shear_ROCK_D2 = _young_ROCK_D2 / 2.0 / (1+ _poiss_ROCK_D2)

; Shear surface material properties
_kn_BOT_shear = 100.0e6 ; base case
_ks_BOT_shear = 50.0e6 ; base case
_cohe_BOT_shear = 400.0e3
_fric_BOT_shear = 19
_tens_BOT_shear = 50.0e3

END

_material_properties

; STAGE # consolidate elastically...

seek
join on ; shear zone is not an interface
change cons = 1 ;<-- material is considered elastic...
prop mat=1 dens= _density_ROCK
gravity 0._grav_NEG,0

DEF _solve_problem
; input: _solve_max_TOTAL_steps, _solve_steps, _solve_ratio_unbal_force
if _solve_max_TOTAL_steps = 0
    _solve_max_TOTAL_steps = 100000
end_if
if _solve_steps = 0

388
_solve_steps = 200
end_if
if _solve_ratio_unbal_force = 0
  _solve_ratio_unbal_force = 0.01 ; <--- 0.01 is 1% and 0.001 is 0.1%
end_if
if _DEBUG_max_counter_loop = 0
  _DEBUG_max_counter_loop = 100000
end_if
command
  step 1
  _store_displacement_instrumentation
end_command
_counter_steps = 0
_flag_keep_looping = 1
_counter_loop = 0
loop while _flag_keep_looping = 1
  _counter_loop = _counter_loop + 1
  _dummy = out("=========================================")
  _dummy = out(' _counter_loop = ' + string(_counter_loop))
  _dummy = out(' _ratio = ' + string(_ratio))
  _dummy = out(' _ratio_TARGET = ' + string(_solve_ratio_unbal_force))
  _dummy = out(' _counter_steps = ' + string(_counter_steps))
  _dummy = out(' _counter_steps_TARGET = ' + string(_solve_max_TOTAL_steps))
  _dummy = out("=========================================")
command
  step _solve_steps
  _store_displacement_instrumentation
end_command
_counter_steps = _counter_steps + _solve_steps
_max_unbal_force = unbal
_max_force = 1e-10
loop i (1, _number_zones)
  _pnt = _z_extra(i,1)
  _sxx = z_sxx(_pnt)
  _syy = z_syy(_pnt)
  _szz = z_szz(_pnt)
  _smean = (_sxx + _syy + _szz)/3.0
  _area = _z_extra(i,2)
  _force = abs(_smean*_area)
  if _force > _max_force
    _max_force = _force
  end_if
end_loop
_ratio = _max_unbal_force/_max_force
if _ratio < _solve_ratio_unbal_force
  _flag_keep_looping = 0
  _line = 'RATIO limit _max_unbal_force/_max_force has been achieved...'
  _dummy = out(_line)
end_if
if _counter_steps > _solve_max_TOTAL_steps
  _flag_keep_looping = 0
  _line = '_solve_max_TOTAL_steps has been achieved...'
  _dummy = out(_line)
end_if
end_loop
END
DEF _fix_boundaries
   _xEast_MIN = _xEast_MIN ; <--- comes from '_read_header_info_topography_blocks' file...
   _xEast_MAX = _xEast_MAX
   _zNorth_MIN = _zNorth_MIN
   _zNorth_MAX = _zNorth_MAX
   _yUp_MIN = _yUp_MIN
   _del_len = 0.1;
   ; --- fix 'y' face...
   _yPOS = _yUp_MIN + _del_len
   _yNEG = _yUp_MIN - _del_len
   command
      bou (_xEast_MIN, _xEast_MAX) (_yNEG, _yPOS) &
      (_zNorth_MIN, _zNorth_MAX) yvel 0.0
   end_command
   ; --- fix 'x' faces...
   _xPOS = _xEast_MIN + _del_len
   _xNEG = _xEast_MIN - _del_len
   command
      bou (_xNEG, _xPOS) (_yNEG, 1e10) &
      (_zNorth_MIN, _zNorth_MAX) xvel 0.0
   end_command
   ; --- fix 'x' faces...
   _xPOS = _xEast_MAX + _del_len
   _xNEG = _xEast_MAX - _del_len
   command
      bou (_xNEG, _xPOS) (_yNEG, 1e10) &
      (_zNorth_MIN, _zNorth_MAX) xvel 0.0
   end_command
   ; --- fix 'z' faces...
   _zPOS = _zNorth_MIN + _del_len
   _zNEG = _zNorth_MIN - _del_len
   command
      bou (_xEast_MIN, _xEast_MAX) (_yNEG, 1e10) &
      (_zNEG, _zPOS) zvel 0.0
   end_command
   ; --- fix 'z' faces...
   _zPOS = _zNorth_MAX + _del_len
   _zNEG = _zNorth_MAX - _del_len
   command
      bou (_xEast_MIN, _xEast_MAX) (_yNEG, 1e10) &
      (_zNEG, _zPOS) zvel 0.0
   end_command
   END

DEF _initialize_pore_pressure
loop iii (1, _number_gpts)
   _pnt = _gpt_extra(iii, 1)
   _pp = _gpt_extra(iii, 2)
   if _pp < 0.0
      _pp = 0.0
   end_if
   gp_pp(_pnt) = _pp
end_loop
END
DEF _initialize_insitu_stresses
  _gamma = _density_ROCK*_grav
  loop iii (1,_number_zones)
    if float(int(iii/100.0)) = iii/100.0
      _line = 'Processing i='+string(iii)+') (of maximum '
    _line = _line + string(_number_zones) + ')' 
    _dummy = out(_line)
  end_if
  _pnt = _z_extra(iii,1)
  _z_xcen = z_x(_pnt)
  _z_ycen = z_y(_pnt)
  _z_zcen = z_z(_pnt)
  ; --- assume a vertical-to-horizontal stress ratio equal to one...
  _x_surf = _z_xcen
  _z_surf = _z_zcen
  _table_num = int(_ntable_TOPOGRAPHY) ; <-- this is the TOPOGRAPHY surface...
  _interpolate_surface
  _y_surf = _y_surf
  _depth = _y_surf - _z_ycen
  _s_vert = -_gamma*_depth ; <-- attention: negative stress is compression...
  _s_horiz = _s_vert * 1.0
  z_sxx(_pnt) = _s_horiz
  z_syy(_pnt) = _s_vert
  z_szz(_pnt) = _s_horiz
  end_loop
END

DEF _compute_maximum_displacement
  _max_disp = -1.0e-40
  _iBlock = block_head
  loop while _iBlock # 0
    _iGp = b_gp(_iBlock)
    loop while _iGp # 0 ; <-- loop through all gridpoints in the block...
      _x_disp = gp_xdis(_iGp)
      _y_disp = gp_ydis(_iGp)
      _z_disp = gp_zdis(_iGp)
      _tot_disp = sqrt(_x_disp*_x_disp+_y_disp*_y_disp+_z_disp*_z_disp)
      if _tot_disp >= _max_disp
        _max_disp = _tot_disp
      end_if
      _iGp = gp_next(_iGp)
    end_loop
    _iBlock = b_next(_iBlock)
  end_loop
END

; Use equivalent 'static' damping...
damp local 0.8 ; <-- this is the default damping for static analyses...
mscale on

;STAGE A - Elastic material
prop mat=1 bulk=_bulk_CONS
prop mat=1 shear=_shear_CONS

_initialize_pore_pressure
_initialize_insitu_stresses
;apply butressing load to toe of reservoir.
hide region 0
bound dip 20 dd 67 org 3830 545 2300 above yrange 525 577 &
stress -1199294 -5462364 -566903 0 0 0 &
ygrad 2093.01 9532.92 989.361 0 0 0
plot dip 0 dd 180 hold
seek
hide region 1
hide region 99
hide region 98
hide region 2
hide region 3
bound dip 13 dd 65 org 3800 550 2300 above yrange 500 573 &
stress -1199294 -5462364 -566903 0 0 0 &
ygrad 2093.01 9532.92 989.361 0 0 0
seek
;

.fix_boundaries
save temporary_6.sav

his reset
his unbal ; his #1
set _solve_max_TOTAL_steps=60000
set _solve_ratio_unbal_force = 0.001
set _solve_steps = 200
.solve_problem

DEF _check_displacement_stage_A
._compute_maximum_displacement
._max_disp = _max_disp
if _max_disp > 10.0 ;
._line = 'Warning STAGE A: _max_disp > 10.0 m during consolidation...'
._dummy = out(_line)
._line = 'Such large movements of gridpoints change configuration of the'
._dummy = out(_line)
._line = 'given problem (e.g., topography, shear surfaces, etc),'
._dummy = out(_line)
._line = 'and problems locating instrumentation may arise...'
._dummy = out(_line)
command
    pause
end_command
end_if
END

._check_displacement_stage_A
._store_displacement_instrumentation

call disp.txt
._avg_vel
d1

save temporary_6a.sav

392
ini xdisp 0
ini ydisp 0
ini zdisp 0
ini xvel 0
ini yvel 0
ini zvel 0

; Stage B1 -- Elastic material
prop mat=1 bulk=_bulk_ROCK_B
prop mat=1 shear=_shear_ROCK_B

his reset
his unbal ; his #1
set _solve_max_TOTAL_steps=60000
set _solve_ratio_unbal_force = 0.001
set _solve_steps = 200
_solve_problem

DEF _check_displacement_stage_B
_compute_maximum_displacement
_max_disp = _max_disp
if _max_disp > 10.0 ;
_line = 'Warning STAGE B: _max_disp > 10.0 m during consolidation...'
_dummy = out(_line)
_line = 'Such large movements of gridpoints change configuration of the'
_dummy = out(_line)
_line = 'given problem (e.g., topography, shear surfaces, etc),'
_dummy = out(_line)
_line = 'and problems locating instrumentation may arise...'
_dummy = out(_line)
command
pause
end_command
end_if
END
_check_displacement_stage_B
_store_displacement_instrumentation
_avg_vel
d1
save temporary_6b.sav

ini xdisp=0
ini ydisp=0
ini zdisp=0
ini xvel=0
ini yvel=0
ini zvel=0

; STAGE B2 -- Elastic material
prop mat=1 bulk=_bulk_ROCK_B2
prop mat=1 shear=_shear_ROCK_B2

his reset
his unbal ; his #1
set _solve_max_TOTAL_steps=60000
set _solve_ratio_unbal_force = 0.001
set _solve_steps = 200
_solve_problem

_check_displacement_stage_B
_store_displacement_instrumentation
_avg_vel
d1
save temporary_6b.sav

ini xdisp=0
ini ydisp=0
ini zdisp=0
ini xvel=0
ini yvel=0
ini zvel=0

;STAGE C -- Elasto-plastic material (with interfaces)
; use joins to define shears
join off
hide region 0
hide region 1
join
seek
hide region 3
hide region 2
join
seek

; --- assign properties to BOTTOM shear zone ---
change rint 0 1 jmat = 1
change rint 1 2 jmat = 1
change rint 2 3 jmat = 1
change rint 3 1 jmat = 1
change jcons=2
prop jmat 1 jkn= _kn_BOT_shear
prop jmat 1 jks= _ks_BOT_shear
prop jmat 1 jfric= _fric_BOT_shear
prop jmat 1 jcohe= _cohe_BOT_shear
prop jmat 1 jtens= _tens_BOT_shear

; --- change material to Mohr-Coulomb ---
change cons = 2 ;<--- change to Mohr-Coulomb elasto-plastic material
prop mat=1 dens= _density_ROCK
prop mat=1 bulk= _bulk_ROCK_C
prop mat=1 shear= _shear_ROCK_C
prop mat=1 bcohe= _cohe_ROCK_C
prop mat=1 phi= _fric_ROCK_C
prop mat=1 btens= _tens_ROCK_C
prop mat=1 psi= _dilat_ROCK_C

his reset
his unbal ; his #1
set _solve_max_TOTAL_steps=60000
set _solve_ratio_unbal_force = 0.001
set _solve_steps = 200
_solve_problem

DEF _check_displacement_stage_C
  _compute_maximum_displacement
  _max_disp = _max_disp
  if _max_disp > 10.0 ;
    _line = 'Warning STAGE C: _max_disp > 10.0 m during consolidation...'
    _dummy = out(_line)
    _line = 'Such large movements of gridpoints change configuration of the'
    _dummy = out(_line)
    _line = 'given problem (e.g., topography, shear surfaces, etc),'
    _dummy = out(_line)
    _line = 'and problems locating instrumentation may arise...'
    _dummy = out(_line)
    command
    pause
    end_command
  end_if
END

_store_displacement_instrumentation

avg_vel

d1
save temporary_6c1.sav

ini xdisp=0
ini ydisp=0
ini zdisp=0
ini xvel=0
ini yvel=0
ini zvel=0

; enable secondary shear surfaces...
seek
join off
join region 0
join region 1
join region 2
join region 3

his reset
his unbal ; his #1
set _solve_max_TOTAL_steps=60000
set _solve_ratio_unbal_force = 0.001
set _solve_steps = 200
_solve_problem

_store_displacement_instrumentation

avg_vel

d1
save temporary_6c2.sav

; load contact pore pressure
ini xdisp=0
ini ydisp=0

395
ini zdisp=0
ini xvel=0
ini yvel=0
ini zvel=0

; STAGE C3 - add contact pp
call load_contact_pp.dat
his reset
his unbal ; his #1
set _solve_max_TOTAL_steps=60000
set _solve_steps = 200
_solve_problem

_check_displacement_stage_C
_store_displacement_instrumentation
_avg_vel
d1
save temporary_6c3.sav

ini xdisp=0
ini ydisp=0
ini zdisp=0
ini xvel=0
ini yvel=0
ini zvel=0

;STAGE D -- change to Erm (d = 0.5) within slide material
change region 1 mat = 2
change region 2 mat = 2
change region 3 mat = 2

prop mat=2 dens= _density_ROCK
prop mat=2 bulk= _bulk_ROCK_D
prop mat=2 shear= _shear_ROCK_D
prop mat=2 bcohe= _cohe_ROCK_D
prop mat=2 phi= _fric_ROCK_D
prop mat=2 btens= _tens_ROCK_D
prop mat=2 psi= _dilat_ROCK_D

his reset
his unbal ; his #1
set _solve_max_TOTAL_steps=60000
set _solve_ratio_unbal_force = 0.001
set _solve_steps = 200
_solve_problem

DEF _check_displacement_stage_D
_compute_maximum_displacement
_max_disp = _max_disp
if _max_disp > 10.0 ;
   _line = 'Warning STAGE D: _max_disp > 10.0 m during consolidation...'
   _dummy = out(_line)
   _line = 'Such large movements of gridpoints change configuration of the'
   _dummy = out(_line)
   _line = 'given problem (e.g., topography, shear surfaces, etc),'
   _dummy = out(_line)
and problems locating instrumentation may arise...

command
pause
end_command
end_if

END

_check_displacement_stage_D
_store_displacement_instrumentation
_avg_vel
d1
save temporary_6d.sav

ini xdisp=0
ini ydisp=0
ini zdisp=0
ini xvel=0
ini yvel=0
ini zvel=0

; STAGE D2 -- change to Erm (d = 0.7) within slide material
prop mat=2 bulk=_bulk_ROCK_D2
prop mat=2 shear=_shear_ROCK_D2
prop mat=2 bcohe=_cohe_ROCK_D2
prop mat=2 phi=_fric_ROCK_D2
prop mat=2 btens=_tens_ROCK_D2

his reset
his unbal ; his #1
set _solve_max_TOTAL_steps=60000
set _solve_ratio_unbal_force = 0.001
set _solve_steps = 200
_solve_problem

_check_displacement_stage_D
_store_displacement_instrumentation
_avg_vel
d1
save temporary_6d2.sav

ini xdisp=0
ini ydisp=0
ini zdisp=0
ini xvel=0
ini yvel=0
ini zvel=0

save stage6_variable_setup.sav
return
B.2.8 Time Step Model

This time steps the model and call all required functions.

[time_steps.dat]

res stage6_variable_setup.sav
call disp.txt
his reset
his unbal ; his #1
call reload_contact_pp.dat
;call check.txt ; this was used during early testing, not for final runs
;his_percent ; this was used during early testing, not for final runs
call stiffness.txt

,******************************
step 200
_apply_stiffness_material
_store_displacement_instrumentation
;_check_disp ; this was used during early testing, not for final runs
_avg_vel
d1
step 200
_apply_stiffness_material
_store_displacement_instrumentation
;_check_disp
_avg_vel
d1
step 200
_apply_stiffness_material
_store_displacement_instrumentation
;_check_disp
_avg_vel
d1
step 200
_apply_stiffness_material
_store_displacement_instrumentation
;_check_disp
_avg_vel
d1
step 200
_apply_stiffness_material
_store_displacement_instrumentation
;_check_disp
_avg_vel
save sav1check1.sav
_avg_vel
d1
_recount_contacts
_refill_contact_pore_pressure_information
;****repeat this section to sav1check40 (40,000 post-consolidation time steps)****
## B.2.9 Load Contact Pore Pressure

This file applies pore pressures to contact elements which define shear surfaces.

### load_contact_pp.dat

```plaintext
table 5000 delete
table 6000 delete
call LOWER_PRE-DRAIN.txt
call UPPER_PRE-DRAIN.txt
```

DEF _determine_number_con

```plaintext
array _contact_extra(3000000,3) ; made way to big
 ;<- 1 column contains the pointer to the contact...
 ;<- 2 column contains Region1....
 ;<- 3 column contains Region2....
LOOP ii (1,3000000)
    _contact_extra(ii,1) = 0
    _contact_extra(ii,2) = 0
    _contact_extra(ii,3) = 0
endloop
_counter = 0
_icontact = contact_head
loop while _icontact # 0
    _isub_con = c_cx(_icontact)
    _block1 = c_b1(_icontact)
    region1 = b_region(_block1)
    _block2 = c_b2(_icontact)
    region2 = b_region(_block2)
    if region2 # region1
        loop while _isub_con # 0 ;<-- loop through all gridpoints in the block...
            _counter = _counter + 1
            _contact_extra(_counter,1) = _isub_con
            _contact_extra(_counter,2) = region1
            _contact_extra(_counter,3) = region2
            _isub_con = cx_next(_isub_con)
        end_loop
    endif
    _icontact = c_next(_icontact)
endloop
_number_con = _counter
END

_determine_number_con
;print _number_con ; for testing
```

DEF _fill_contact_pore_pressure_information

```plaintext
loop ii (1, _number_con)
    _isub_con = _contact_extra(ii,1)
    z = cx_z(_isub_con)
    x = cx_x(_isub_con)
    y = cx_y(_isub_con)
    _x_surf = x
```
_z_surf = _z
region1 = _contact_extra(ii,2)
region2 = _contact_extra(ii,3)
if region1 # region2
  if region1 = 0
    _table_num = 5000
  endif
  if region2 = 0
    _table_num = 5000
  endif
  if region1 = 3
    if region2 = 2
      _table_num = 6000
    endif
    if region2 = 1
      _table_num = 6000
    endif
  endif
  if region2 = 3
    if region1 = 2
      _table_num = 6000
    endif
    if region1 = 1
      _table_num = 6000
    endif
  endif
  if region1 = 2
    if region2 = 1
      _table_num = 6000
    endif
    if region2 = 2
      if region1 = 1
        _table_num = 6000
      endif
    endif
  endif
endif
_interpolate_surface
_y_surf = _y_surf
_depth = _y_surf - _y
_pp = _depth*gamma_water
if _pp >= 0
  cx_pp(isub_con) = _pp
endif
if _pp < 0
  cx_pp(isub_con) = 0
endif
end_loop
END
_fill_contact_pore_pressure_information
B.2.10 Re-Load Contact Pore Pressure

This file re-loads contact pore pressure. This is required periodically because as joint slip occurs new contact elements are created and the default pore pressure on new elements is zero.

reload_contact_pp.dat

```
table 5000 delete
table 6000 delete
call LOWER_PRE-DRAIN.txt
call UPPER_PRE-DRAIN.txt

DEF _determine_number_con
array _contact_extra(3000000,3)  ; made way to big
    ;<-- 1 column contains the pointer to the contact...
    ;<-- 2 column contains Region1....
    ;<-- 3 column contains Region2....

LOOP ii (1,3000000)
    _contact_extra(ii,1) = 0
    _contact_extra(ii,2) = 0
    _contact_extra(ii,3) = 0
endloop

_counter = 0
_icontact = contact_head
loop while _icontact # 0
    _isub_con = c_cx(_icontact)
    _block1 = c_b1(_icontact)
    region1 = b_region(_block1)
    _block2 = c_b2(_icontact)
    region2 = b_region(_block2)
    if region2 # region1
        loop while _isub_con # 0 ; <-- loop through all gridpoints in the block...
            _counter = _counter + 1
            _contact_extra(_counter,1) = _isub_con
            _contact_extra(_counter,2) = region1
            _contact_extra(_counter,3) = region2
            _isub_con = cx_next(_isub_con)
        end_loop
    endif
    _icontact = c_next(_icontact)
endloop

_number_con = _counter
END

_determine_number_con
print _number_con

DEF _fill_contact_pore_pressure_information
loop ii (1,_number_con)
```

401
_isub_con = _contact_extra(ii,1)
_z = cx_z(_isub_con)
_x = cx_x(_isub_con)
_y = cx_y(_isub_con)
_x_surf = x
_z_surf = z
region1 = _contact_extra(ii,2)
region2 = _contact_extra(ii,3)
if region1 # region2
    if region1 = 0
        _table_num = 5000
        endif
    if region2 = 0
        _table_num = 5000
        endif
    if region1 = 3
        if region2 = 2
            _table_num = 7000
            endif
        if region2 = 1
            _table_num = 6000
            endif
        endif
    if region2 = 3
        if region1 = 2
            _table_num = 7000
            endif
        if region1 = 1
            _table_num = 6000
            endif
        endif
    if region1 = 2
        if region2 = 1
            _table_num = 6000
            endif
        endif
    if region2 = 2
        if region1 = 1
            _table_num = 6000
            endif
        endif
    endif
_endloop
END
_fill_contact_pore_pressure_information
B.2.11 Vary Joint Stiffness

This file adjusts the joint stiffness parameters based on location of joint elements if they fall in a thick, thin or average region (Figure 3.17).

**stiffness.txt**

```plaintext
prop jmat 2 jkn=_kn_BOT_shear
prop jmat 2 jks=_ks_BOT_shear
prop jmat 2 jfric=_fric_BOT_shear
prop jmat 2 jcohe=_cohe_BOT_shear
prop jmat 2 jtens=_tens_BOT_shear

DEF _stiff_param
    _kn3 = _kn_BOT_shear*5
    _kn1 = _kn_BOT_shear*0.2
    _ks3 = _ks_BOT_shear*5
    _ks1 = _ks_BOT_shear*0.2
END

_stiff_param
prop jmat 1 jkn=_kn1
prop jmat 1 jks=_ks1
prop jmat 1 jfric=_fric_BOT_shear
prop jmat 1 jcohe=_cohe_BOT_shear
prop jmat 1 jtens=_tens_BOT_shear

prop jmat 3 jkn=_kn3
prop jmat 3 jks=_ks3
prop jmat 3 jfric=_fric_BOT_shear
prop jmat 3 jcohe=_cohe_BOT_shear
prop jmat 3 jtens=_tens_BOT_shear

call Thickness_multiplier_tables.txt

DEF _apply_stiffness_material
    _icontact = contact_head
    loop while _icontact # 0
        _isub_con = c_cx(_icontact)
        _block1 = c_b1(_icontact)
        region1 = b_region(_block1)
        _block2 = c_b2(_icontact)
        region2 = b_region(_block2)
        if region2 # region1
            loop while _isub_con # 0 ; <-- loop through all gridpoints in the block...
                _z = cx_z(_isub_con)
                _x = cx_x(_isub_con)
                _y = cx_y(_isub_con)
                _x_surf = _x
                _z_surf = _z
                _table_num = 20000
                _interpolate_surface
                _y_surf = _y_surf
                _mat_number = 3 ; high stiffness
```

403
if _y_surf < 2.5 ; normal stiffness
    _mat_number = 2
endif
if _y_surf < 1.5 ; low stiffness
    _mat_number = 1
endif
cx_mat(_isub_con) = _mat_number
_isub_con = cx_next(_isub_con)
end_loop
endif
_iconact = c_next(_icontact)
endloop
END

B.2.12 Check for Steady State

This file was developed to initially test how many time steps were reasonable to achieve steady state deformation rates.

check.txt

DEF _check_disp
    _x = 0
    _y = 0
    _z = 0
    _x_prev = 0
    _y_prev = 0
    _z_prev = 0
loop kk (1,_num_instr)
    ii = 1
    _npts = 1
    _pnt_zone_interp = _data_instr(kk,1,4)
    _id_table_xdisp = int(_data_instr(kk,1,5))
    _id_table_ydisp = int(_data_instr(kk,1,6))
    _id_table_zdisp = int(_data_instr(kk,1,7))
    _tab_pos = table_size(_id_table_xdisp)
    _curr_x_disp = ytable(_id_table_xdisp,_tab_pos)
    _tab_pos_prev = _tab_pos - 1
    _prev_x_disp = ytable(_id_table_xdisp,_tab_pos_prev)
    _tab_pos = table_size(_id_table_ydisp)
    _curr_y_disp = ytable(_id_table_ydisp,_tab_pos)
    _tab_pos_prev = _tab_pos - 1
    _prev_y_disp = ytable(_id_table_ydisp,_tab_pos_prev)
    _tab_pos = table_size(_id_table_zdisp)
    _curr_z_disp = ytable(_id_table_zdisp,_tab_pos)
    _tab_pos_prev = _tab_pos - 1
    _prev_z_disp = ytable(_id_table_zdisp,_tab_pos_prev)
    _x = _x + _curr_x_disp
    _y = _y + _curr_y_disp
    _z = _z + _curr_z_disp
    _x_prev = _x_prev + _prev_x_disp

404
\_y\_prev = \_y\_prev + \_prev\_y\_disp \\
\_z\_prev = \_z\_prev + \_prev\_z\_disp 
end\_loop \\
curr\_total = sqrt(\_x\_x + \_y\_y + \_z\_z)/\_num\_instr \\
prev\_total = sqrt(\_x\_prev\_x + \_y\_prev\_y + \_z\_prev\_z)/\_num\_instr \\
\_diff = curr\_total - prev\_total \\
\_percent = \_diff/prev\_total*100 \\
if \_percent < 0.05 \\
\_checker = \_checker + 1 \\
else \\
\_checker = 0 \\
endif \\
command \\
print \_percent \\
print \_checker \\
endcommand \\
if \_checker = 10 \\
\_junk = out('equilibrium has been acheived') \\
command \\
\pause \\
endcommand \\
endif \\
END \\

B.2.13 Check for Exploding Nodes 

This file is used to control "exploding nodes" which occur when tensile failure occurs near or at the topographic surface.

disp.txt 

DEF \_avg\_vel \\
\_total\_vel = 0 \\
\_iBlock = block\_head \\
\_junk = 0 \\
loop while \_iBlock \# 0 \\
if b\_region(\_iBlock) = 1 \\
\_zi = b\_zone(\_iBlock) \\
loop while \_zi \# 0 \\
loop gp (1,4) \\
\_ig = z\_gp(\_zi,\_gp) \\
\_gp\_x\_vel = gp\_x\_vel(\_ig) \\
\_gp\_y\_vel = gp\_y\_vel(\_ig) \\
\_gp\_z\_vel = gp\_z\_vel(\_ig) \\
\_gp\_x = \_gp\_x\_vel*\_gp\_x\_vel \\
\_gp\_y = \_gp\_y\_vel*\_gp\_y\_vel \\
\_gp\_z = \_gp\_z\_vel*\_gp\_z\_vel \\
\_gp\_vel = sqrt(\_gp\_x + \_gp\_y + \_gp\_z) \\
\_total\_vel = \_gp\_vel + \_total\_vel \\
\_junk = \_junk + 1 \\
endloop
_zi = z_next(_zi)
endloop
endif
_iblock = b_next(_iBlock)
endloop
_average = _total_vel/_junk
END

DEF d1

count = 0
count2 = 0
count3 = 0
_iBlock = block_head
_junk = 1
loop while _iBlock # 0
if b_region(_iblock) # 0
 _zi = b_zone(_iblock)
 _junk = 0
 min_d = 10000
 max_d = 0
 _total_vel = 0
loop while _zi # 0
 loop gp (1,4)
   _ig = z_gp(_zi, gp)
   _gpxd = gp_xvel(_ig)
   _gpyd = gp_yvel(_ig)
   _gpzd = gp_zvel(_ig)
   _gpx = _gpxd*_gpxd
   _gpy = _gpyd*_gpyd
   _gpz = _gpzd*_gpzd
   _gpvel = sqrt(_gpx + _gpy + _gpz)
   _total_vel = _gpvel + _total_vel
   _junk = _junk + 1
   if _gpvel < min_d
      min_d = _gpvel
   endif
   if _gpvel > max_d
      max_d = _gpvel
   endif
endloop
 _zi = z_next(_zi)
endloop
vel_average = _total_vel/_junk
_zi2 = b_zone(_iblock)
loop while _zi2 # 0
 loop gp (1,4)
   _ig = z_gp(_zi2, gp)
   _gpxd = gp_xvel(_ig)
   _gpyd = gp_yvel(_ig)
   _gpzd = gp_zvel(_ig)
   _xxl = gp_x(_ig) - 0.1
   _xxh = gp_x(_ig) + 0.1
   _yyl = gp_y(_ig) - 0.1
   _yyh = gp_y(_ig) + 0.1
   _zzl = gp_z(_ig) - 0.1
   _zzh = gp_z(_ig) + 0.1
406
\_gpx = \_gpxd*\_gpxd
\_gpy = \_gpyd*\_gpyd
\_gpz = \_gpzd*\_gpzd
\_gpvel = sqrt(\_gpx + \_gpy + \_gpz)
if \_gpyd > 0
  if \_gpvel > (5000*vel\_average)
    count = count + 1
    gp_xvel(\_ig) = 0
    gp_yvel(\_ig) = 0
    gp_zvel(\_ig) = 0
    command
      bound xrange \_xxl,\_xxh yrange \_yyl,\_yyh zrange \_zzl,\_zzh yload = -100000
    endcommand
  endif
endif
if \_gpvel > \_average*100
  count2 = count2 + 1
  gp_xvel(\_ig) = 0
  gp_yvel(\_ig) = 0
  gp_zvel(\_ig) = 0
  if \_gpyd > 0
    count3 = count3 + 1
    command
      bound xrange \_xxl,\_xxh yrange \_yyl,\_yyh zrange \_zzl,\_zzh yload = -100000
    endcommand
  endif
endif
endloop
\_zi2 = z\_next(\_zi2)
endloop
\_iBlock = b\_next(\_iBlock)
endloop
ii = out('count' + string(count))
ii = out('count2' + string(count2))
ii = out('count3' + string(count3))
END

B.2.14 Data Files

These are all of the data files called during model runs. The entire files are quite long, and have been truncated here as the main purpose of this section is to demonstrate format rather than provide data.

ELEV_XYZ[100]_[3DEC_geom].dat
;This file generates a 3DEC DEM (digital elevation model)

poly prism a &
  0 0 0 &
  0 0 100 &
  0 1897.086594 100 &
This file summarizes information of blocks in file ELEV_XYZ_[500]_3DEC_geom.dat

2601 ; number of lines in this file
0 ; this is x_min (East+)
5000 ; this is x_max (East+)
0 ; this is z_min (North+)
5000 ; this is z_max (North+)
0 ; this is y_min (Upward+)
1910 ; this is y_max (Upward+)
50 ; number of blocks in x (East+) direction
50 ; number of blocks in z (North+) direction
51 ; number of gpts (blocks+1) in x (East+) direction
51 ; number of gpts (blocks+1) in z (North+) direction
; The following lines contain East, North, min elev., max elev.
0 0 0 1901.000002
0 100 0 1897.086594
0 200 0 1872.140104
0 300 0 1857.160731
0 400 0 1841.279859
0 500 0 1829.092159
0 600 0 1805.224146
0 700 0 1778.248234
0 800 0 1744.714903
0 900 0 1723.501426
Input table files are used to define topography, shear zone geometries, the location of water tables and the spatial distribution of shear zone stiffness values. As an example the format of lower_pre-drain.txt (lower water table at pre-drain levels.) is shown below. Table x000 (in this case table 5000) holds a number of table markers for the east-west dimension of a model on 20 m 20 m intervals. Tables in the form 500x-to 5251 give elevation data along each the north-south model dimension (again in 20 m intervals) for each of the east-west intervals.

table 5000 5001 0
table 5000 5002 20
table 5000 5003 40
table 5000 5004 60
table 5000 5005 80
table 5000 5006 100
:
    table 5000 5249 4960
    table 5000 5250 4980
    table 5000 5251 5000

    table 5001 0 1930.4
    table 5001 20 1924.8
    table 5001 40 1919.1
    table 5001 60 1913.4
    :
    table 5001 4980 561.9
    table 5001 5000 556.5
    :
    table 5251 5000  -39.0
B.3 Model Sensitivity Testing

B.3.1 Grid Resolution and Mesh Density Sensitivity Testing

Sensitivity testing has been completed to assess the influence of grid size and mesh density. Grid size effectively defines the resolution of the shear surface; a smaller grid size allows for smaller-scale curvature. Mesh density influences block deformation; as the number of elements in a block increases, more complex deformation can be achieved. It is ideal to have the smallest possible grid size, particularly when assessing the influence of small-scale variability between shear surface geometries, in combination with small mesh elements to improve model deformability. However, small grid size and high mesh density, demand higher computer memory availability and lengthens the model runtimes, so sensitivity testing has been completed for optimization.

For varying grid and mesh size, models are compared based on total displacements achieved prior to the landslide reaching equilibrium, where equilibrium is defined by displacement over the last 2000 time steps less than 5% of total achieved displacements over the entire model run. For testing, the grid sizes included 90, 100, 130, 150 and 200 meters, and mesh densities were assessed at 1/6, 1/5, 1/4, 1/3 and 1/2 of each grid size. Figure B1 demonstrates that as grid size is reduced from 200 to 90 m, the grid resolution appears to no longer influence the total magnitude of displacement for grid sizes less than 100 m, therefore 100 m grid resolution has been selected. As mesh size is reduced, the total displacements increase due to reduced model stiffness. Based on this testing there is no optimized mesh-size, where zones a size smaller than a given threshold, no longer influence results. Instead a mesh density of 1/4 the grid resolution (25 m) was selected to
ensure that more than three zones span each grid element, while not limiting memory and incurring excessive model run times.

**Figure B.1**: Average surface displacements achieved at a set time step interval for varying grid and mesh sizes.

### B.3.2 Sensitivity Testing of Downie Slide Shear Surface Strength Parameters (Friction and Cohesion)

Sensitivity testing has been completed to assess the influence of joint friction and cohesion on simulated Downie Slide behaviour. Friction values ranging between 15° and 23° on 2° increments were tested, and cohesion was varied between 100 kPa and 900 kPa on 200 kPa increments. Figure B.2 illustrates the total displacement achieved for a constant number of numerical time steps. It is apparent that slide displacements increase significantly for cohesion at 100 and 300 kPa, and friction less than 17°. Based on these
results and the assumption that the slide mass is active, but slow moving, a friction value of 19° and cohesion of 400 kPa have been selected as shear surface strength parameters for Downie Slide numerical simulations.

Figure B.2: Sensitivity testing results for varying shear surface cohesion and friction values.

**B.4 References**