THE INFLUENCE OF
HEALED INTRABLOCK ROCKMASS STRUCTURE
ON THE BEHAVIOUR OF DEEP EXCAVATIONS
IN COMPLEX ROCKMASSES

by

Jennifer Jane Day

A thesis submitted to the Department of Geological Sciences and Geological Engineering

In conformity with the requirements for

the degree of Doctor of Philosophy

Queen’s University
Kingston, Ontario, Canada
December, 2016

Copyright © Jennifer Jane Day, 2016
Abstract

Conventional rockmass characterization and analysis methods for geotechnical assessment in mining, civil tunnelling, and other excavations consider only the intact rock properties and the discrete fractures that are present and form blocks within rockmasses. Field logging and classification protocols are based on historically useful but highly simplified design techniques, including direct empirical design and empirical strength assessment for simplified ground reaction and support analysis. As modern underground excavations go deeper and enter into more high stress environments with complex excavation geometries and associated stress paths, healed structures within initially intact rock blocks such as sedimentary nodule boundaries and hydrothermal veins, veinlets and stockwork (termed intrablock structure) are having an increasing influence on rockmass behaviour and should be included in modern geotechnical design. Due to the reliance on geotechnical classification methods which predate computer aided analysis, these complexities are ignored in conventional design. Given the comparatively complex, sophisticated and powerful numerical simulation and analysis techniques now practically available to the geotechnical engineer, this research is driven by the need for enhanced characterization of intrablock structure for application to numerical methods. Intrablock structure governs stress-driven behaviour at depth, gravity driven disintegration for large shallow spans, and controls ultimate fragmentation.

This research addresses the characterization of intrablock structure and the understanding of its behaviour at laboratory testing and excavation scales, and presents new methodologies and tools to incorporate intrablock structure into geotechnical design practice. A new field characterization tool, the Composite Geological Strength Index, is used for outcrop or excavation face evaluation and provides direct input to continuum numerical models with implicit rockmass structure. A brittle overbreak estimation tool for complex rockmasses is developed using field observations. New methods to evaluate geometrical and mechanical properties of intrablock structure are developed. Finally, laboratory direct shear testing protocols for interblock structure are critically evaluated and extended to intrablock structure for the purpose of determining input parameters for numerical models with explicit structure.
Co-Authorship

The thesis “The Influence of Healed Intrablock Rockmass Structure on the Behaviour of Deep Excavations in Complex Rockmasses” is the product of research conducted by the author, Jennifer Jane Day. Although scientific and editorial feedback was provided by Dr. Mark Diederichs and Dr. Jean Hutchinson, the written content is solely that of the author.
Acknowledgements

Firstly, I would like to thank my supervisors, Dr. Mark Diederichs and Dr. Jean Hutchinson, for their tremendous support, inspiration, and guidance. Their passion, dedication, and the experience they bring to teaching and research is unparalleled, and I am lucky to have had the opportunity to learn from them. They have provided me with more opportunities than I could have ever imagined, have challenged me to be the best I can be, and have had great faith in me to succeed. Mark and Jean, I honestly don’t know where I would be without you both. Thank you for being exceptional mentors and great friends.

This research was funded by the Natural Sciences and Engineering Research Council of Canada (NSERC), the Centre for Excellence in Mining Innovation (CEMI), the Nuclear Waste Management Organization (NWMO) of Ontario, Canada, the Swedish Nuclear Fuel and Waste Management Company (SKB), the Government of Ontario’s Research Fund and Graduate Scholarship, the Railway Ground Hazard Research Program that includes support by CN Rail and Canadian Pacific, and multiple scholarships from Queen’s University. Particular thanks go to Tom Lam and Mark Jensen from NWMO for their support of this research.

I am indebted to the generosity, kindness, and enthusiasm for research in the people and companies I encountered when doing field work, including Alex Calderon and Antofagasta Minerals S.A., the GeoBlast team led by Iván Zapata, Glencore’s Sudbury Integrated Nickel Operations and the geology team led by Steve Falconer, Brad Simser at Glencore, and the El Teniente Division of Codelco. I would like to thank Peter Goguen at GCTS for his invaluable technical support for the direct shear test system in the Queen’s Geomechanics Group, as well as Scott, Norbert, and Shawn at Danton Machine & Welding Inc. for their instrumental help with drilling laboratory test samples. To Denis Labrie from CanmetMINING, Natural Resources Canada, thank you for the insightful discussions on laboratory testing. To Kathy Kalenchuk at Mine Design Engineering, thank you for the professional development opportunities that have helped to shape my field work skills.

The world-class geological laboratory equipment in the department has been essential to the success of this research. I would like to thank the professors in the department of Geological Sciences and Geological Engineering who were kind enough to grant me access to their equipment: Professors Ron Peterson, Gema Olivo, Dan Layton-Matthews, Kurt Kyser, and Georgia Fotopoulos. Of course, I would
never have learned how to use the equipment so effectively without the expert guidance from Agatha, April, Jordan, Christabel, Brian, Steve, Jenn, Rebecca, and Mike on numerous occasions. To Professors Ron Peterson, Kurt Kyser, Rob Harrap, John Hanes, Noel James, and Laurent Godin, thank you for the occasional discussions and words of wisdom over the years on both research and teaching.

A special thanks to Dr. Evert Hoek, Dr. Derek Martin, Dr. Erik Eberhardt, Dr. Vassilis Marinos, Dr. Paul Marinos, and Rick Lovat for the inspirational discussions on the world of geomechanics and its future directions in academia and industry.

It has been a privilege to be part of the camaraderie of the Queen’s Geomechanics Group during the past five years. To the past (Chrysothemis, Megan, Matthew, Shaun, Jeff, Ehsan, Gabe, Michelle, Matt, Connor, Cortney, Cara, Steve, Anna, Dani, and Colin) and present (Michelle, Felipe, Kevin, Kiarash, Ioannis, Brad, Sarah, Andrew, Daniela, Eric, Mike, Neda, Jonathan, Emily, and Ryan) members who have shared this journey with me, thank you for the local and international adventures, being a fabulous support network for research, and enriching our lab space. I am very grateful for our friendships and I hope we stay in touch.

I would also like to thank my other graduate student colleagues in the department for keeping me on the straight and narrow whenever I applied geological science to geomechanics engineering.

To my lab assistants, Connor, Wesley, Liam, Katy, Alex, and Rebecca, thank you for your great effort and attention to detail at any hour of the day, which has certainly enhanced the quality of this research.

To Becky, Catherine, Meredith, and Michelle, thank you for listening, encouraging, and occasionally talking some sense into me. You have been incredible friends on my journey so far, and I am excited for our adventures to come. To Jean, Mark, Katy, Liz, and Shadow, thank you for being my second (Kingston) family. For teaching me skills of practice, perseverance, and performance that I have frequently called upon during this thesis, I owe many years of thanks to my trumpet teachers, Dan Tremblay and Paul Sanvidotti.

Finally, and most importantly, thank you to my family for their love, support, and guidance in life. Mom and Dad, thank you for providing me with endless opportunities to pursue my passions and a sage perspective on life. The diversity of talents in my extended family is inspirational and continues to challenge me to succeed; thank you to all the Days, Sievenpipers, Sukornyks, Suttons, and Bilous’s.
Dedication

To my family
Statement of Originality

I hereby certify that all of the work described within this thesis is the original work of the author. Any published (or unpublished) ideas and/or techniques from the work of others are fully acknowledged in accordance with the standard referencing practices.

Jennifer Jane Day

December, 2016
### Table of Contents

Abstract .......................................................................................................................... ii
Co-Authorship ............................................................................................................... iii
Acknowledgements ..................................................................................................... iv
Dedication .................................................................................................................. vi
Statement of Originality ........................................................................................... vii
List of Figures ........................................................................................................... xiv
List of Tables ............................................................................................................. xxxi
List of Abbreviations .............................................................................................. xxxiv

Chapter 1 Introduction ............................................................................................... 1
  1.1 Purpose of Study ............................................................................................... 1
  1.2 Empirical Geotechnical Design Practices ......................................................... 4
  1.3 Numerical Geotechnical Design Practices ......................................................... 6
  1.4 Research Objectives ....................................................................................... 8
  1.5 Thesis Outline and Contributions ................................................................... 9
  1.6 References ..................................................................................................... 13

Chapter 2 Numerical Methods, the Hoek-Brown Strength Criterion, and the Geological Strength Index .................................................................................. 15
  2.1 Rockmass Classification .................................................................................. 15
  2.1.1 Difficulties with Multiple Structural Elements ............................................. 16
  2.2 Numerical Methods ..................................................................................... 17
  2.2.1 Finite Element Method (FEM) .................................................................. 18
  2.2.2 Explicit Structural Elements .................................................................... 21
  2.2.3 Networks of Structural Elements ............................................................... 22
  2.2.4 Implementation of the Hoek-Brown Strength Criterion and Geological Strength Index .................................................. 24
  2.3 Development of the Hoek-Brown Strength Criterion and Geological Strength Index .................................................. 24
    2.3.1 Quantifications of GSI ......................................................................... 42
  2.4 References .................................................................................................... 48

Chapter 3 A New Composite GSI Approach and Applications to Intrablock Structure .......................................................... 51
  3.1 Introduction .................................................................................................... 51
  3.2 Applying GSI to Multiple Sets or Suites of Structure ...................................... 54
  3.3 Influence of Intrablock Structures on Rockmass Strength .............................. 57
  3.4 Incorporating Intrablock Structure into Equivalent-Continuum Numerical Design .......................................................... 60
  3.5 Accounting for Intrablock Structure using GSI .............................................. 60
# Chapter 4 Brittle Overbreak Prediction for Hydrothermal Complex Rockmasses with Healed Structure

## 4.1 Introduction and Background

## 4.2 Predicting Brittle Spalling Overbreak

### 4.2.1 Empirical Spalling Prediction

### 4.2.2 Mechanistic Interpretation of Spalling Prediction

## 4.3 Drift Overbreak Observations at El Teniente Mine

### 4.3.1 Station 2

### 4.3.2 Station 3

### 4.3.3 Station 4

### 4.3.4 Station 5

## 4.4 Overbreak Profile Measurements

### 4.4.1 Geological Descriptions of Units

#### 4.4.1.1 Stockwork Mafic Complex

#### 4.4.1.2 Dacite Porphyry

#### 4.4.1.3 Anhydrite Breccia

#### 4.4.1.4 Diorite Porphyry

### 4.4.2 Hydrothermal Alteration of Units

---

### References

---

---
Chapter 9 Discussion and Conclusions

9.1 Discussion

9.1.1 Characterization of Complex Rockmasses for Numerical Models with Implicit Structure

9.1.2 Brittle Overbreak Estimation for Excavations in Complex Rockmasses

9.1.3 Characterization of Complex Rockmasses for Numerical Models with Explicit Structure

9.1.4 Laboratory Evaluation of Intrablock Structure for Numerical Models with Explicit Structure

9.1.5 Limitations of Current Research

9.2 Summary of Conclusions

9.2.1 Characterization of complex rockmasses for numerical models with implicit structure

9.2.2 Brittle overbreak estimation for excavations in complex rockmasses

9.2.3 Characterization of complex rockmasses for numerical models with explicit structure

9.2.4 Laboratory evaluation of intrablock structure for numerical models with explicit structure

9.3 Future Research

9.4 Contributions

9.4.1 Articles Published in Refereed Journals

9.4.2 Articles Submitted for Review

9.4.3 Articles in Preparation

9.4.4 Fully Refereed Conference Papers (Day as first author only)
9.4.5 Refereed Extended Abstract and Presentation (no paper) ................................................................. 369
9.4.6 Reports Produced from Research Activities ......................................................................................... 370
9.4.7 Posters Produced from Research Activities ........................................................................................ 370
9.4.8 Invited Presentations .......................................................................................................................... 370
9.5 References .......................................................................................................................................... 371

Appendix A Direct Shear Test Sample Photographs ................................................................................. A-1
Appendix B Direct Shear Test Data ........................................................................................................... B-1
Appendix C X-Ray Diffraction Sample Preparation Procedure .............................................................. C-1
Appendix D MLA Mineral Reference Library for the Cobourg Limestone .............................................. D-1
List of Figures

Figure 1-1: Examples of modern excavations in complex rockmasses where intrablock structure can influence rockmass behaviour; (top) Olmos trans-Andean water transport tunnel (Diederichs et al., 2013); (middle) Ontario’s deep geological repository for low to intermediate level nuclear waste storage (courtesy of NWMO); (bottom) 3-Dimensional model of the El Teniente block cave mine showing undercut levels with hundreds of kilometers of excavation infrastructure (modified after Pardo et al., 2012) ................................................................. 2

Figure 1-2: Examples of intrablock structure; (a) drill core with hydrothermal pink gypsum and white quartz veins; (b) drill core with hydrothermal quartz veins with variable thicknesses from 0.5 mm to 7 cm; (c) hydrothermal quartz veins at an excavation face (rock bolts and plates for scale); (d) sample cube of Cobourg limestone with intrablock structure defined as tortuous clay-rich layers between calcite-rich nodules (cube dimensions are 40 cm); (e) unrolled scan of cylindrical core surface of Cobourg limestone showing more detail of intrablock structure (dark grey layers) ................................................................. 3

Figure 1-3: Adjustment factor for MRMR block strength to account for veins within the rock block (modified after Read and Stacey, 2009; Laubscher and Jakubec, 2001) ................................................................. 6

Figure 1-4: Summary diagram describing the thesis research activities and objectives herein to geotechnical design of complex rockmasses ................................................................................................................................. 9

Figure 2-1: Range of numerical methods used in geomechanics from continuum to discontinuum codes 18

Figure 2-2: FEM shape function for a six-noded triangular mesh element (after Pande et al., 1990) .... 20

Figure 2-3: Goodman joint fracture element for FEM models (Goodman et al., 1968) ............. 22

Figure 2-4: Range of Voronoi polygon shape regularity available in Phase² (modified after RocScience, 2015) ......................................................................................................................................................... 23

Figure 2-5: Correlations of m and s to rockmass classification systems Q and RMR for Panguna Andesite data (Hoek and Brown, 1980). This relationship was used to approximate m and s values for various rockmasses shown in Table 2-2 ......................................................................................................................... 26

Figure 2-6: Approximate relationship between rockmass quality and Hoek-Brown material constants (Hoek and Brown, 1988) ......................................................................................................................... 29

Figure 2-7: Estimation of $m_b/m_i$ and a based on rock structure and surface condition (Hoek et al., 1992) 32

Figure 2-8: Estimation of constants $m_b/m_i$, s, a, Young’s modulus (E), Poisson’s Ratio (ν), and GSI for the Generalized Hoek-Brown strength criterion based on rockmass structure and discontinuity surface conditions. Note that the values given in this table are for an undisturbed rockmass (Hoek et al., 1995) . 35
Figure 2-9: Contoured field estimation chart of Geological Strength Index (GSI) based on geological descriptions (Hoek and Brown, 1997)

Figure 2-10: Most recent qualitative GSI chart with added massive and laminated/sheared categories (Hoek and Marinos, 2000)

Figure 2-11: Brief description and schematic diagrams of tunnel behaviour types used in the Tunnel Behaviour Chart that is shown in Figure 2-12 (Marinos, 2012)

Figure 2-12: Tunnel Behaviour Chart (TBC): An assessment of rockmass behaviour in tunnelling from experience in Greece (Marinos, 2012)

Figure 2-13: Generalized Hoek-Brown composite strength envelopes using the DISL approach for brittle failure of Lac du Bonnet granite, where the laboratory-measured tensile strength is assumed to anchor the damage initiation and systematic damage envelopes (Diederichs, 2007)

Figure 2-14: Block volume calculation components in a block with three joint sets (Cai et al., 2004)

Figure 2-15: Quantified GSI chart (modified after Cai et al., 2004)

Figure 2-16: Updated GSI chart by Hoek et al. (2013), where the contour grid has been linearized, and the structure and surface condition scales can be quantified with user-defined axes (scales A and B)

Figure 2-17: Appropriate use and limitations of GSI depending on scale (Hoek et al., 2013)

Figure 3-1: Three examples of hydrothermal vein types of intrablock structure from Chile

Figure 3-2: Geological Strength Index (GSI) chart, modified after the Tzamos and Sofianos (2007) version (with quantifications by Cai et al., 2004 and descriptions by Hoek and Marinos, 2000), showing an example of a complex rockmass with suites of interblock (blue square at JB₁, BB₁) and intrablock (orange diamond at JB₂, BB₂) structures; the conventional worst case GSI (white circle with purple outline) and new Composite GSI (CGSI) (green filled circle at JB*, BB*) ratings of this rockmass are also shown in their calculated (JB, BB) coordinates

Figure 3-3: Surface of GSI chart based on JB and BB integer coordinates compared to fitted and contoured Composite GSI (CGSI) power function

Figure 3-4: Rock from the Oyu Tolgoi Cu-Au Porphyry deposit in Mongolia. Joints (arrows) and intrablock structure (other visible traces) in core (left) and in underground rockmass (middle); ultimate fragmentation (right) shows size controlled by intrablock structure under disturbance (courtesy M. S. Diederichs)

Figure 3-5: Drill core from Chilean porphyry and Sudbury, Canada magmatic deposits showing various infill - wall rock contact qualities of intrablock structure; (a-c) strengthening welded quartz veins; (d-f) sulphide veins (pyrite, chalcopyrite) with some quartz; (g-i) healed and broken gypsum veins; (j) epidote vein that broke during drilling; (k) weak calcite vein that broke during drilling; (l-m) weak swelling clay infilling that has been altered and expanded by water application during core logging
Figure 3-6: New GSI chart for complex rockmasses that contain interblock and intrablock structures. The added column is used to describe the infill quality of strengthening intrablock structure and descriptions of other intrablock structure have been added to existing columns. A summary of equations to calculate the Composite GSI (CGSI) is also provided.

Figure 3-7: Region of expected brittle rockmass behaviour indicated on new GSI chart for complex rockmasses that contain interblock and intrablock structures, where brittle rockmass failure criteria should be applied in numerical analyses.

Figure 3-8: Illustrative summary of FEM comparison of conventional GSI and CGSI.

Figure 3-9: Example of anhydrite vein (Courtesy of Codelco Div. El Teniente).

Figure 3-10: Finite-Element (RS2 by RocScience (2015)) model geometries of the circular 6 m-diameter excavation showing a) full model of implicit rockmass structure with mesh, external boundaries, and the four query measurement lines, inset (i) shows mesh detail near excavation; b) quarter model plus full excavation with explicit interblock (joint) geometry, inset (ii) shows structure geometry and mesh detail; c) quarter model plus full excavation with explicit intrablock (vein) geometry, inset (iii) shows structure geometry and mesh detail; d) quarter model plus full excavation with explicit interblock (joint) and intrablock (vein) geometries (i.e. full rockmass), inset (iv) shows structure geometry and mesh detail; all models have a far field section of an equivalent continuum region for computational stability, and the external boundaries have zero displacement (i.e. pinned) conditions.

Figure 3-11: Graphs of elastic calibration for joints highlighting Trial 8 which was selected as the best fit to the implicit GSI model based on standard deviation analysis. Total displacements in the explicit models are normalized to those of the implicit model and stiffness units are in GPa/m.

Figure 3-12: Graphs of elastic calibration for veins highlighting Trial 15 which was selected as the best fit to the implicit GSI model based on standard deviation analysis. Total displacements in the explicit models are normalized to those of the implicit model and stiffness units are in GPa/m.

Figure 3-13: Total displacements of equivalent continuum (GSI and CGSI) models normalized to the model with explicit structure.

Figure 3-14: Graph of plastic model calibration for joint strength by measuring depth of plastic yield around the excavation.

Figure 3-15: Graph of plastic model calibration for vein strength by measuring depth of plastic yield around the excavation.

Figure 3-16: Calibrated plastic models showing depth of yield measurements from excavation boundary.

Figure 3-17: Depth of yield of plastic equivalent continuum (GSI and CGSI) models normalized to the model with explicit structure.
Figure 3-18: Geometry of explicit structure suites for each model relative to the excavation sizes for both cases ................................................................. 84

Figure 3-19: GSI charts for each case example (A and B) showing individual structure suites as coloured circular dots and sequential CGSI values for the rockmass in each model (numbers highlighted with blue squares). The legends for the coloured dots are in Table 3-16 and Table 3-18 .......................... 85

Figure 3-20: FEM models (A1 to A4) of Case A showing deformed $\sigma_1$ contours, yielded material (mesh) elements, and yielded joint elements. Deformation scale factor is 10 ......................................................... 87

Figure 3-21: FEM models (B1 to B4) of Case B showing deformed $\sigma_1$ contours, yielded material (mesh) elements, and yielded joint elements. Deformation scale factor is 2 .......................................................... 89

Figure 3-22: Direct shear results on a deeper limestone unit similar to the Lindsay Formation. The lower bound peak and residual strengths are used to determine the strength of the nodular intrablock structure (modified after NWMO, 2011) ................................................................................................... 89

Figure 3-23: Excavation faces along the adit at various depths below ground surface .......................... 92

Figure 3-24: Stereonet showing observed adit and joint set orientations (right-hand strike/dip) ............ 93

Figure 3-25: Histogram of selected block sizes as measurement of average spacing of quartz veins ...... 93

Figure 3-26: Site observations from the 600 m deep excavation face of the adit at El Teniente. (a) Approximate excavation profile of adit approximately 5 m behind the face; (b) view of excavation face including immediate roof; (b) detailed view of excavation face with highlighted joint planes (4 sets) and average quartz vein spacing defined by the fragmented block size of the excavated material .......................... 94

Figure 3-27: In situ stresses at El Teniente (Diederichs, 2016) ................................................................................................................................. 96

Figure 3-28: Relevant quadrant of FEM model with explicit structure, and a detailed inset of explicit structure and adit dimensions ........................................................................................................ 97

Figure 3-29: FEM model results of the 600 m deep excavation face of the adit at El Teniente, comparing maximum principal stresses ($\sigma_1$) and yielded elements, for the explicit and GSI equivalent continuum models; depth of yield measurements in the roof are indicated for each model .............................................. 99

Figure 3-30: Estimated depth of yield measurements (normalized to 3 m tunnel radius) from FEM models at a range of excavation depths show a better fit between the explicit and CGSI solutions when compared to the conventional worst case and joints only GSI approaches. The nonlinear data curves are best fits of the Carreau-Yasuda rheological model. The explicit rockmass structure and equivalent continuum GSI values in the FEM models are based on observations at the 600 m deep adit excavation face. The in situ stress conditions vary with depth (refer to Figure 3-27). The inset excavations show yielded material elements for the 100 and 2000 m deep worst-case equivalent continuum GSI models. The overbreak observations at excavation faces at 450, 550, and 600 m depths are best approximated using the CGSI equivalent continuum models .................................................................................................................. 101
Figure 3-31: Hoek-Brown strength envelopes for an altered andesite in a porphyry copper deposit, for failures through intact rock (solid lines) and hydrothermal vein intrablock structure (dashed lines); the inset photo shows an example of drill core with hydrothermal veins ....................................................... 105

Figure 3-32: FEM model results illustrate the implications of using intact material properties from matrix or structural laboratory testing failure modes on a 10 m diameter tunnel at a range of depths ............... 107

Figure 4-1: Boundary element elastic solution of induced stresses around a circular excavation, showing a highly stressed zone in the roof when the maximum principal stress, $\sigma_1$, is horizontal and perpendicular to the excavation axis ............................................................................................................................... 115

Figure 4-2: Diagram of progressive formation of notch in roof of an excavation by brittle failure in highly deviatoric in situ stresses (Martin et al., 1997) ........................................................................................................................................ 115

Figure 4-3: Empirical estimation tool for spalling depth around an excavation (CI is the minimum strength for crack initiation) (modified after Diederichs, 2007, 2010) ...................................................... 118

Figure 4-4: Composite damage initiation - spalling limit (DISL) brittle constitutive model (solid line) synthesis of investigations into spalling mechanics; low confinement spalling limit is threshold between crack initiation strength limit in low confinement and crack damage / long term yield strength limit at high confinement; graphics illustrate crack behaviours in laboratory and in situ conditions (Diederichs, 2007) ........................................................................................................................................ 119

Figure 4-5: Composite plot of mechanistic numerical simulations of spalling around an excavation using the DISL approach, compared to the empirical best fit mean and limits from Figure 4-3; depth of failure calculations using the conventional GSI rockmass (shear) approach for GSI illustrate fundamental difference between brittle and plasticity based strength criteria (Diederichs, 2007) ........................................................................................................................................ 121

Figure 4-6: Excavation Damage Zone (EDZ) (modified after Ghazvinian, 2015) ................................... 122

Figure 4-7: The empirical depth of failure linear fit from Diederichs (2007) superimposed by the upper 68% prediction intervals of numerically based EDZs that represent the maximum EDZ depth, compared to in situ case histories of EDZ measurements (Perras and Diederichs, 2015) ............................................................... 124

Figure 4-8: 3-Dimensional model of the El Teniente mine, showing the undercut levels (modified after Pardo et al., 2012) ........................................................................................................................................ 125

Figure 4-9: Panel and pre-undercut caving methods at the El Teniente mine (Brzovic and Villaescusa, 2007) ........................................................................................................................................ 126

Figure 4-10: Overview of overbreak observations around excavations of different orientations in the New Mine Level undercut level at the El Teniente copper porphyry mine on October 16th, 2014. Larger overbreak was observed as notch geometries in directions sub-perpendicular to the maximum principal stress (e.g. Stations 4a, 4b, and 5b), while smaller overbreak was generally observed parallel to the maximum principal stress (e.g. Stations 2b, 3b, and 5a) ........................................................................................................................................ 128
Figure 4-11: Overbreak observations at Station 2 in the New Mine Level undercut level at El Teniente on October 16th, 2014. Drift heading 2a parallel to $\sigma_2$ has notch overbreak, while 2b drift parallel to $\sigma_1$ has a rounded overbreak profile.

Figure 4-12: Overbreak observations at Station 3 in the New Mine Level undercut level at El Teniente on October 16th, 2014. Drift heading is parallel to $\sigma_1$ and has rounded overbreak profile. The detail view of the unsupported roof at the heading shows partially formed brittle spall features parallel to the excavation boundary.

Figure 4-13: Diagram of field-scale boundary-parallel spalling by Diederichs (1999) that illustrates the boundary parallel brittle stress fractures observed in the field.

Figure 4-14: Overbreak observations at Station 4 in the New Mine Level undercut level at El Teniente on October 16th, 2014. The shape and extent of overbreak profiles changes at different orientations along the ramp with respect to principal stress orientations. Irregular notch profiles form when the drift is parallel to $\sigma_2$ (as in 4a, 4d, and 4e) while rounded profiles occur when the drift is parallel to $\sigma_1$ (as in 4b, 4c, and 4f).

Figure 4-15: Overbreak observations at Station 5 in the New Mine Level undercut level at El Teniente on October 16th, 2014.

Figure 4-16: Plan view of the undercut level, New Mine Project at El Teniente showing lithologies and drift layout. The case study drifts A and B are highlighted in red.

Figure 4-17: Overbreak profiles along the undercut level drift A (parallel to $\sigma_1$) in various lithologies.

Figure 4-18: Range of overbreak along drift A through (a) all units, and (b-e) the individual units.

Figure 4-19: Overbreak profiles along undercut level drift B (subparallel to $\sigma_2$) in stockwork mafic complex geological unit in the New Mine Level at El Teniente.

Figure 4-20: Range of overbreak along drift B entirely through the stockwork mafic complex unit.

Figure 4-21: Examples of lithologies studied at El Teniente: Stockwork mafic complex (a) diabase, (b) gabbro, and (c) basalt porphyries; (d) dacite porphyry; (e) anhydrite breccia with clasts from the stockwork mafic complex unit; and (f) diorite porphyry (photographs courtesy of Codelco).

Figure 4-22: Examples of early alteration phase vein types in the New Mine Level at El Teniente; (a) quartz ± anhydrite, biotite, chalcopyrite, molybdenum, bornite, pyrite, chloride (without halo), (b) quartz-biotite-anhydrite-chlorite ± chalcopyrite, pyrite, biotite (silica or silica-chlorite halo with adjacent disseminated biotite); (c) chalcopyrite and/or bornite and/or pyrite and/or molybdenite ± anhydrite, quartz, biotite (without halo); (d) magnetite ± quartz, biotite (silica halo) (courtesy of Codelco).

Figure 4-23: Intact and failed laboratory UCS sample pairs of three lithologies at El Teniente (Courtesy of M. S. Diederichs).
Figure 4-24: Empirical linear fit for brittle failure prediction of homogeneous massive to moderately jointed rockmasses after Diederichs (2007, 2010) compared to new maximum depth of failure cases observed at the El Teniente undercut level (drifts A and B) in four lithologies and two orientations; the poor correlation is attributed to the heterogeneous character of the El Teniente rockmasses ................. 146

Figure 4-25: Ranges of overbreak depths for case drifts A and B at the El Teniente New Mine Level Project. Means and standard deviations for each lithology are plotted along with the absolute maximum overbreak measurements (see Figure 4-18 and Figure 4-20). Data is compared to the upper 68% depth of failure prediction intervals of numerically based EDZs by Perras and Diederichs (2015) that represent the maximum EDZ depth when compared to in situ measurements, as well as the empirical linear fit by Diederichs (2010) and after Martin et al. (1999). The overbreak measurements from El Teniente vary by lithology with respect to the existing prediction functions due to the complex intrablock structures present ............................................................ 148

Figure 4-26: Mechanically brecciated contact zone where the dacite porphyry intruded the stockwork mafic complex that was previously altered and veined, in (a) an excavation wall and (b) drill core from nearby the studied New Mine Level zone (modified after Skewes et al., 2002) ........................................ 151

Figure 4-27: Ranges of overbreak depths for case drifts A and B at the El Teniente New Mine Level Project, including new failure prediction ranges for these heterogeneous rockmasses defined by functions listed in Table 4-7. The range of “Brecciated Units” includes the anhydrite breccia and the brecciated stockwork mafic complex – dacite porphyry contact units. The empirical linear fit by Diederichs (2010) and after Martin et al. (1999) is included for reference ................................................................. 152

Figure 4-28: Stress-driven structurally controlled failure where stress fractures accumulate parallel to the excavation boundary and migrate along widely spaced, persistent joints. The result is an extremely deep failure not otherwise found in massive spalling notch formation (Diederichs et al., 2013) .............. 153

Figure 5-1: Typical shear stress-deformation relationships for various types of interblock structures (after Goodman, 1969) ........................................................................................................................................ 163

Figure 5-2: Normal compression of an extension fracture in a granodiorite sample, where $V_{mc}$ is the maximum possible closure, and P shows the yield point of the sample, showing a hyperbolic behaviour. The difference between compression curves A and B describes the compression of the mated joint (independent of the system stiffness), as shown in the right graph, and likewise for curves A and C for the non-mated joint (Goodman, 1976) ........................................................................................................ 164

Figure 5-3: Photos of hand samples (1 to 4) selected for thin section analysis and use in this study ...... 166

Figure 5-4: Composite photos of veins (samples 1 to 4) in cross-polarized, transmitted light thin section. The length of all scale bars represents 5 mm in the thin sections ................................................................. 167
Figure 5-5: Schematic of FEM models of UCS test simulations showing geometry, boundary, and loading conditions, for the two methods of modelling veins: by explicit vein materials and a calibrated, equivalent numerical joint element ................................................................. 169
Figure 5-6: Composite photos of veins and corresponding FEM explicit vein models in Phase 2 ............. 171
Figure 5-7: Stress-strain profiles of UCS tests of each sample (1 to 4) with both wall rock Young’s moduli values (37,500 and 75,000 MPa). The vein samples with real modal mineralogies are compared to samples of veins with 100% of each mineral present, in order to compare the relative influence of minerals on the overall axial stress-strain behaviour. Due to the ideal nature of numerical models, the pre-peak elastic portions of the curves are linear ................................................................. 173
Figure 5-8: Yield progression of the explicitly modelled sample 4 vein, with a Young’s modulus value of 75,000 MPa, that contains 60% pyrite, 37% muscovite and 3% quartz ........................................ 176
Figure 5-9: Diagram of the general model geometry used for all tunnel models ......................... 178
Figure 5-10: Detailed view of the Phase 2 FEM tunnel model showing the anisotropic Voronoi vein network ........................................................................................................................................ 178
Figure 5-11: Major principal stress (σ₁, MPa), total displacement (m), and yielded element results for models 1-6 (samples 1 and 2) ................................................................. 181
Figure 5-12: Major principal stress (σ₁, MPa), total displacement (m), and yielded element results for models 7-12 (samples 3 and 4) ................................................................. 182
Figure 6-1: Bi-linear (Patton, 1966) and non-linear (Barton and Choubey, 1977) criteria for rough joint shear strength .................................................................................................................................. 190
Figure 6-2: (a) Drill core section of wall rock with quartz vein; (b) primary (pre-peak), secondary (post-peak), and tertiary (ultimate) considerations of intrablock structure (e.g. vein) stiffness and strength .... 191
Figure 6-3: Schematic of general and idealized geometry used for all models, including in-situ stress conditions ......................................................................................................................................... 194
Figure 6-4: (a) Photo of drift heading in case study area with major veins traced in yellow; (b) example detail of composite FEM model from this study ............................................................................................................ 195
Figure 6-5: Model results of primary (first model) and secondary-tertiary (second model) states for three representative FEM models, with contours of maximum principal stress (σ₁); detail of model excavation geometry is shown at the bottom-right .................................................................................. 196
Figure 6-6: Representative cross-sections of case study drift showing overbreak around arched excavations, and relative frequency of occurrence along the 340 m section of drift, based on 69 profile measurements ...................................................................................................................................... 197
Figure 7-1: The Geological Strength Index (GSI) chart, modified after Hoek et al. (2013), showing estimates of the GSI ratings for the interblock (square), intrablock (diamond), conventional GSI approach
Figure 7-2: Generalized Hoek-Brown (Hoek et al. 2002) strength envelopes reflecting the various GSI ratings for the hypothetical example discussed in Figure 7-1. Equations that define the Hoek-Brown criterion are included in the figure to show the role of GSI.

Figure 7-3: Classification of rock sample failure mode during testing (modified after Marambio et al. 1999).

Figure 7-4: Example measurement of vein block size. Veins are measured from the top inflection point, and only veins that are visible around the full circumference of the drill core are included.

Figure 7-5: 20 m drill core section of a partially leached andesitic tuff that was logged using the discussed core logging methods; data collected from this core was used for the conveyor tunnel numerical analysis.

Figure 7-6: Strength envelopes of intact rock (H-B) and three vein types (M-C) used in models, including both peak and residual properties. Peak intact strength envelope is a regression fit from tensile, UCS and triaxial test data.

Figure 7-7: Schematic of the general model geometry used for all models.

Figure 7-8: Stereonet analysis of vein orientations in Dips software by RocScience (2013a), where the strike/dip of the mean set planes are oriented at 332/57° and 047/74°.

Figure 7-9: Detailed view of FEM model 4b that contains two anisotropic Voronoi vein networks.

Figure 7-10: Overview of models showing progressive detail and major principal stress (σ₁) contours. (Note 'Pinwheel' effect in model 4b due to vein network anisotropy)

Figure 7-11: Major principal stress (σ₁), total displacement, and yielded element results for all conveyor tunnel models in hydrothermally altered andesite.

Figure 7-12: 15 m drill core section of a nodular limestone that was logged using the discussed core logging methods. Data collected from this core was used for the numerical analysis of the DGR.

Figure 7-13: Peak and residual strength envelopes for intact rock (H-B) and intrablock structure (H-B converted to M-C) used in models. Peak intact strength envelope is a regression fit from tensile, UCS and triaxial test data, and peak intrablock strength is the minimum boundary of the laboratory test data.

Figure 7-14: (left) Schematic of the general and idealized model geometry used for all DGR models; (right) detail of a composite Finite Element Model (FEM) model with explicit rockmass structure.

Figure 7-15: Major principal stress (σ₁), total displacement, and yielded element results for all models.

Figure 8-1: Examples of strategies to include explicit sedimentary intrablock structure in a FEM model at the excavation scale; interblock bedding is represented in both models with horizontal persistent structure.
elements (orange); intrablock structure can be represented by (left) non-persistent horizontal parallel
structure elements or (right) Voronoi element geometry ................................................................. 234
Figure 8-2: Drill core of Cobourg limestone from the Bowmanville quarry ........................................ 235
Figure 8-3: Illustration of a Canadian design for a Deep Geological Repository (Noronha, 2016) ....... 238
Figure 8-4. Geological cross-section through the Michigan Basin with approximately 45x vertical
exaggeration (modified after Gartner Lee Ltd., 2008a) ................................................................. 240
Figure 8-5: The St. Marys Cement Bowmanville quarry near Bowmanville, Ontario, Canada, is the source
location of the Cobourg limestone for this direct shear laboratory testing program; a) geographical site
context; b) aerial view of quarry with precise source location indicated at location d,e (image courtesy of
Google); c) view of source location; d) quarry wall at source location; e) rock block pile is source of
laboratory testing samples (photographs courtesy of M. Diederichs) ............................................. 241
Figure 8-6: Cross-section of proposed overall final slope geometry at the St. Marys Cement Bowmanville
quarry; source level of Cobourg samples used for direct shear testing program is shown (courtesy of St.
Mary’s Cement) .................................................................................................................................. 242
Figure 8-7: Examples of Cobourg limestone blocks from the Bowmanville quarry, showing the cross-
section of intrablock structure on the vertical surfaces; the wet patterns on the bottom two blocks
appeared during drying and highlight the uptake of water by the intrablock structure ......................... 244
Figure 8-8: (a) Full scale drill pattern geometrically optimized to accommodate testing plan for 2”, 3” and
4” diameter cores; (b) drill pattern traced onto first Cobourg limestone block; (c) unique drill pattern for
block developed with consideration of partially formed fracture plane to maximize intact drill cores; (d)
drill pattern and fracture trace marked onto second Cobourg limestone block. ................................ 245
Figure 8-9: (a) Using Kitchen-Walker 4’ radial drill to extract a 3” diameter core out of a Cobourg
limestone block; (b) Lifting remaining block, using an overhead crane and friction fit strap, off drill
planform to collect exposed cylindrical cores ...................................................................................... 247
Figure 8-10: Packing process used for each individual piece of cylindrical core to preserve samples during
transport from drilling site at Danton Drilling & Welding Inc. to Geomechanics testing laboratory at
Queen’s University ............................................................................................................................. 247
Figure 8-11: X-ray diffraction results of the baseline test on the Cobourg limestone from the Bowmanville
quarry .................................................................................................................................................. 250
Figure 8-12: X-ray diffraction results of the sample fractions after digestion in hydrochloric acid and the
clay separation procedure .................................................................................................................. 251
Figure 8-13: Detail view of the X-ray diffraction results of the sample fractions after digestion in
hydrochloric acid and the clay separation procedure; the change in counts at the primary peak for illite
and muscovite at ~10° 20 are highlighted here .................................................................................... 252
Figure 8-14: Detail view of the X-ray diffraction results comparing the < 2 μm fraction of the calcite digested Cobourg limestone sample before and after treatment with ethylene glycol..............................253
Figure 8-15: SEM images of Cobourg limestone thin sections from the Bowmanville quarry (30 m b.g.s.) ..................................................................................................................................................259
Figure 8-16: SEM image compilation of fossil-controlled crack propagation in a thin section of the Cobourg limestone from the Bowmanville quarry (30 m b.g.s.) ...............................................................260
Figure 8-17: Example of fossil defining part of pre-existing fracture surface in drill core sample of Cobourg limestone from the Bowmanville quarry ......................................................................................261
Figure 8-18: Photograph comparing intrablock structure character between drill core samples of Cobourg limestone from the St. Marys Cement Bowmanville quarry and Bruce DGR boreholes .................263
Figure 8-19: Graphical comparison of MLA mineralogy results for Bruce DGR borehole samples ......265
Figure 8-20: Relationship between mineral composition of major constituent minerals from the Bowmanville quarry and Bruce DGR borehole source locations, illustrated here with respect to change in depth below ground surface ........................................................................................................................266
Figure 8-21: Large-scale Precambrian to Paleozoic tectonic elements in southern Ontario (modified after Gartner Lee Ltd., 2008b; after Johnson et al., 1992) .........................................................................................................................267
Figure 8-22: SEM image compilation of fossil-controlled crack propagation in a thin section of the Cobourg limestone from borehole DGR-5 at 732.20 m b.g.s. ......................................................................................................................268
Figure 8-23: SEM images of the Cobourg limestone from borehole DGR-5 showing the general grain distributions and cracks preferentially travelling through the fine grained matrix, with undulation caused by intersections with larger mineral grains (most commonly calcite) ........................................................................................................................................269
Figure 8-24: GCTS direct shear testing system at the Queen’s Geomechanics Testing Laboratory .......271
Figure 8-25: Cylinder scanner used to capture the unrolled circumferential surface of a specimen (designed by Wesley Dossett, NSERC USRA) ..................................................................................................................................................272
Figure 8-26: Image from cylinder core scanner of the unrolled circumferential surface of 3 inch diameter core B1-3-4; the fracture surface between segments A and B was measured to determine the orientation of the fracture surface as a best fit ellipse with respect to the core axis for direct shear sample preparation; inset detail shown in Figure 8-27 ..................................................................................................................................................273
Figure 8-27: Inset detail of unrolled core surface from cylindrical core scan shown in Figure 8-26 ......274
Figure 8-28. (a) Original photographic scan of circumferential core surface; (b) photographic levels adjusted to amplify darker and lighter regions of core surface with selections of fracture and intact target shear planes; (c) sinusoidal curve fit of target shear plane ..................................................................................................................................................274
Figure 8-29. (a) Cobourg direct shear sample being scanned by NextEngine desktop 3-Dimensional laser scanner on rotating platform; (b) Photograph draped over 3D scan of fracture surface of direct shear sample; (c) 3D point cloud of sample generated by scanner ................................................................. 276
Figure 8-30: Camera setup for digital photograph collection; photogrammetry turntable at left and top right; vertical photos taken with colour correction palette for post-processing ........................................ 277
Figure 8-31: Photogrammetry model of the lower half of a direct shear sample after testing; photogrammetry models were created using the PhotoScan software package (Agisoft, 2016) .............. 278
Figure 8-32. (a) Diamond saw used for cutting rock cores; (b) detail view of diamond saw blade aligned with designated drill plane ........................................................................................................ 279
Figure 8-33. (a) Elliptical shear plane determined by circumferential intersection of sinusoidal best fit curve on paper wrapped around core; (b) Saw patterns marked parallel to elliptical target shear plane at 2.5-3 cm on each side of fracture plane; (c) 1 cm wide grout boundary marked parallel to elliptical target shear zone with a pre-existing fracture surface ................................................................. 279
Figure 8-34: New procedure for intrablock sample preparation to draw target shear zone on sample; (a-b) sinusoidal best fit of target shear surface ellipse, printed to scale and cut into bottom and top halves; (c) wrapping scaled sinusoidal best fit graph around sample; (d-f) tracing sine pattern onto sample surface so it is evenly distributed around the specifically targeted trace line of intrablock structure, with a spacing of 1.1 mm for the shear zone; here, the permanent marker used for tracing was approximately 0.5 mm thick, reducing the final zone to 1 cm ........................................................................................................ 280
Figure 8-35: Example of a targeted intrablock structure shear surface with elliptical traces from the sinusoidal best fit curve, the 1 cm thick target shear zone (grout boundary), and alignment targets for photogrammetry model processing ........................................................................................................ 281
Figure 8-36: Cement grout materials and mixing process .............................................................................. 282
Figure 8-37: (a) alignment of intrablock sample in shear ring; (b) bottom boundary of target shear zone is levelled with the top surface of the sample ring with a first pass by eye and confirmation by measurement with a level; (c) pouring grout into bottom sample ring while being careful to not get grout on the sample; (d) grout is filled to be flush with sample ring without spilling over; (e) 1 cm thick layer of plasticine is formed onto grout surface; (f) top sample ring is installed and held in place with black collar ring while grout is poured and cures ........................................................................................................ 283
Figure 8-38: Sequence of preparation of direct shear sample rings; (a) bottom sample encased in grout and stainless steel ring; (b) detail view of grout surface aligned 0.5 cm below target shear plane; (c) top of 1 cm thick blue plasticine layer and upper steel ring in place; (d) fully prepared direct shear sample with grout in upper half and shear plane encased with temporary black collar; (e) shear sample in direct shear
machine with collar removed and sand level used to have smooth contact with cap; (f) sample covered by cap that couples with normal and shear loads during test ................................................................. 284
Figure 8-39: Legend for direct shear sample identification numbers; Fbc refers to the interblock fracture surface between intact core pieces b and c of the 40 cm total length of 2 inch diameter core # 4 from block B1, which was used in the second 2 inch diameter fracture shear test at 0.5 MPa normal stress; I1 refers to the first target intrablock structure shear zone in the 3 inch diameter core # 1 from block B2, which was used in the second 3 inch diameter intrablock shear test at 8 MPa ................................................................. 285
Figure 8-40: Testing schedule of the direct shear test program ........................................................................ 287
Figure 8-41: Example linear fits to three cycles of a fracture direct shear test that has a maximum normal stress of 8 MPa, with the semi-logarithmic and linear closure laws shown in the left and right graphs, respectively ................................................................................................................................. 291
Figure 8-42: Compilation of stiffness characteristics for all Cobourg limestone direct shear tests .......... 292
Figure 8-43: Stiffness characteristics of pre-existing fractures and intrablock structure in the Cobourg limestone compared to the granite fracture database from Zangerl et al. (2008) ................................................................. 293
Figure 8-44: Compilation of normal stiffness for all Cobourg limestone samples, including both linear and semi-log closure law results. The semi-logarithmic normal stiffness is the product of the stiffness characteristic and maximum normal stress for each sample ................................................................................ 294
Figure 8-45: Normal stiffness results for Cobourg samples with a pre-existing fracture, including both linear and semi-logarithmic closure law results. The semi-log normal stiffness is the product of the stiffness characteristic and maximum normal stress for each sample ........................................................................ 295
Figure 8-46: Normal stiffness results for intrablock structure Cobourg samples, including both linear and semi-log closure law results. The semi-logarithmic normal stiffness is the product of the stiffness characteristic and maximum normal stress for each sample ................................................................................ 296
Figure 8-47: Quantitative statistical comparison using the coefficient of determination ($R^2$) to measure the variance of the linear regressions between the linear and semi-logarithmic closure laws to the normal stiffness data. This analysis suggests that the linear closure law is a better measure of normal stiffness for these direct shear test results ................................................................. 297
Figure 8-48: Sample qualitative statistical comparison of the linear fit regular residuals between the linear and semi-logarithmic closure laws of cycle 1 shown in Figure 8-41; the linear closure law residual plot is more randomly dispersed around the horizontal axis when compared to the semi-logarithmic law, which indicates a better fit to the data. Note the $R^2$ values for the sample data agree with the residuals analysis ................................................................................................................................. 298
Figure 8-49: Laboratory and in situ normal closure data of granite. The red dashed threshold indicates maximum normal stress of approximately 20 MPa where non-linear behaviour fully develops (modified after Zangerl et al., 2008).

Figure 8-50: Measurement types for shear stiffness used in this study. Peak and yield types are found in the literature and the best fit chord type is a new approach developed in this testing program.

Figure 8-51: Literature data of shear stiffness results from direct shear tests that has been compiled for this investigation to analyze global trends.

Figure 8-52: Composite graph of shear stiffness results of 2-inch diameter fracture and intrablock samples.

Figure 8-53: Composite graph of shear stiffness results of 3-inch diameter fracture and intrablock samples.

Figure 8-54: Ratios of shear stiffness between 3 inch and 2 inch sample diameters; the left graph is the ratio between shear stress stiffness while the right graph is the ratio between shear load stiffness, which removes the surface area component of stress.

Figure 8-55: Outlier shear stiffness test (left) compared to the other 2 inch fracture test which has a typical result (right).

Figure 8-56: Example images of fracture shear surfaces before testing showing representative examples of roughness and undulation character; approximate shear directions are indicated by white arrows; (a-c) 2 inch diameter samples as 3D point clouds from a LiDAR scan; (d-f) 3 inch diameter samples in oblique photographs.

Figure 8-57: 3-Dimensional block model of Cobourg sample block, showing consistent undulations with ~3 inch wavelength across the entire block width.

Figure 8-58: Comparison of Cobourg (chord) shear stiffness results to literature (peak and yield) shear stiffness data.

Figure 8-59: Bruce DGR Mark II placement room design layout for sedimentary host rock (Radakovic-Gunzina et al., 2015); the panel access tunnel is modelled in this study.

Figure 8-60: Finite element model geometry of DGR panel access tunnel in RS2 (RocScience, 2015).

Figure 8-61: Detail of excavation with bedding and intrablock explicit structure and mesh in RS2 (RocScience, 2015).

Figure 8-62: Total displacement around the excavation for each model with yielded structural elements in red.

Figure 8-63: Comparison of total displacements in the excavation roof, moving upward away from the excavation boundary.
Figure 8-64: Example measurements of peak, residual, and ultimate shear strengths for a good quality 2 inch diameter intrablock sample that was tested at a normal stress of 2 MPa; (a) overhead photo of the bottom half of the sample after shearing; (b) overhead photo of the top half of the sample after shearing; (c) an oblique view of the bottom half of the sample; (d) shear stress – shear displacement results that are used to measure shear stiffness and shear strength properties; (e) normal displacement – shear displacement results that are used to measure the post-peak dilation angle (dilation angle is discussed in the next section) .................................................................................................................................................. 320

Figure 8-65: All shear strength results and best fit Mohr-Coulomb envelopes of the 2 inch diameter samples prior to discarding samples that experienced grout interference; the samples to be removed during data filtering are highlighted in red ........................................................................................................................................... 321

Figure 8-66: All shear strength results and best fit Mohr-Coulomb envelopes of the 3 inch diameter samples prior to discarding samples that experienced grout interference; the samples to be removed during data filtering are highlighted in red ........................................................................................................................................... 322

Figure 8-67: Shear strength results of the 2 inch diameter samples that have been filtered to exclude post-peak results that experienced grout interference ........................................................................................................................................... 323

Figure 8-68: Shear strength results of the 3 inch diameter samples that have been filtered to exclude post-peak results that experienced grout interference ........................................................................................................................................... 324

Figure 8-69: Examples of grout interference with post-peak shear behaviour: (a-b) 3 inch diameter intrablock sample B1-3-1-I3-3a tested at 3 MPa normal stress, where (a) is the top half and (b) is the bottom half; (c-d) 2 inch diameter fracture sample B1-2-4-Fab-3b tested at 3 MPa normal stress, where (c) is the top half and (d) is the bottom half. Zones of grout disturbance are indicated by the dotted lines in (b) and (c)........................................................................................................................................... 325

Figure 8-70: Summary of Mohr-Coulomb strength properties of all failure envelopes from the filtered data sets........................................................................................................................................... 326

Figure 8-71: Example of the dilation angle measurement of a 3 inch fracture sample tested at 1.2 MPa normal stress (sample ID: A1-3-13-Fab-1.2a). The dilation angle is measured as the linear slope of normal displacement - shear displacement immediately after peak strength. The measured slope is indicated by green dashed line. The measured dilation angle was corrected based on the dip of the shear surface, which was measured on the sample after testing........................................................................................................................................... 329

Figure 8-72: Photos of sample A1-3-13-Fab-1.2a after testing; (a) vertical view of bottom half of sample, which remained stationary during testing; (b) vertical view of top half of sample, which sheared to the right (toward the black arrow on the sample ring) with respect to the bottom half; (c) oblique view of bottom half of sample showing fracture surface does not intersect grout; (d) oblique view of top half of sample with plasticine showing fracture surface does not intersect grout ........................................................................................................................................... 330
Figure 8-73: Three views of an example 3-Dimensional point cloud model of a direct shear sample in PolyWorks (InnovMetric, 2015) that was used to measure the orientation of the shear surface relative to sample ring (horizontal reference plane) ................................................................. 331

Figure 8-74: Dilation angle measurements of the Bowmanville quarry Cobourg limestone direct shear samples; Q2 (median) values are reported in the box plot ................................................................. 332

Figure 8-75: Photographs of core samples selected for direct shear testing at Canmet that are discussed in this investigation .................................................................................................................. 334

Figure 8-76: Comparison of Canmet normal stiffness data to 3 inch fracture sample results from the Bowmanville quarry Cobourg limestone ........................................................................................................... 335

Figure 8-77: Comparison of Canmet peak shear stiffness data to 3 inch sample results from the Bowmanville quarry Cobourg limestone ........................................................................................................... 336

Figure 8-78: Comparison of Canmet peak and residual shear strength data to 3 inch sample results from the Bowmanville quarry Cobourg limestone ........................................................................................................... 337

Figure 9-1: New GSI chart for complex rockmasses that contain intrablock structure. The added column is used to describe the infill quality of strengthening intrablock structure and descriptions of other intrablock structure have been added to existing columns. A summary of equations to calculate the Composite GSI (CGSI) is also provided .................................................................................................................. 348

Figure 9-2: Ranges of overbreak depths for case drifts A and B at the El Teniente New Mine Level project. Means and standard deviations are plotted along with the absolute maximum overbreak measurements. Data is compared to the mechanistic prediction intervals by Perras and Diederichs (2015) for different parts of the excavation damage zone (EDZ) and linear empirical fit by Diederichs (2010) and after Martin (1999). The overbreak measurements from El Teniente vary by lithology with respect to the existing prediction functions due to the complex intrablock structures and other heterogeneous rocks that are present .................................................................................................................................. 351

Figure 9-3: Ranges of overbreak depths for case drifts A and B at the El Teniente New Mine Level Project, including new failure prediction ranges for these heterogeneous rockmasses defined by functions listed in Table 4-7. The range of “Brecciated Units” includes the anhydrite breccia and stockwork mafic complex – dacite porphyry contact units. The empirical linear fit by Diederichs (2010) and after Martin et al. (1999) is included for reference .................................................................................................................................. 352

Figure 9-4: Intrablock structure property calibration at the laboratory test sample scale using thin section analysis; (top left) hand sample source of thin section of pyrite and muscovite vein; (bottom left) Thin section of vein with geometry of mineral grain boundaries constructed in FEM numerical model; (right) schematic of UCS simulation used to calibrate joint element stiffness and strength properties to axial stress-strain response of the discrete vein material model .................................................................................................................................. 354
Figure 9-5: (a) Drill core section of wall rock with quartz vein; (b) primary (pre-peak), secondary (post-peak), and tertiary (ultimate) considerations of intrablock structure stiffness and strength .................... 355

Figure 9-6: Unrolled image of drill core surface of the Cobourg limestone that was tested in direct shear, showing the heterogeneous nature of the calcite-rich nodules and clay-rich intrablock structure............. 359
List of Tables

Table 1-1: Inverse values of Mohs hardness for use in MRMR block strength to account for weakening of the rockmass by veins (Laubscher and Jakubec, 2001) .......................................................... 6
Table 2-1: Geometrical parameters for selected joint network models in Phase² (RS2) (after RocScience, 2015) ................................................................................................................................. 23
Table 2-2: Approximate equations for principal stress relationships and Mohr envelopes for five categories of intact rock and jointed rockmasses (redrafted from Hoek and Brown, 1980) ................. 27
Table 2-3: Values of the Hoek-Brown constant m for intact rock. Note that values in parenthesis are estimates (Hoek and Brown, 1997) .................................................................................................................. 33
Table 3-1: Modified Joint Condition rating, Modified JCond ss, to include intrablock structure .......... 65
Table 3-2: Geometry of rockmass structure in terms of GSI parameters ............................................. 67
Table 3-3: Intact rock properties in terms of the Generalized Hoek-Brown criterion ......................... 70
Table 3-4: Joint stiffness properties tested in elastic model trial runs ............................................... 72
Table 3-5: Standard deviation analysis of normalized total displacements in elastic joint model trial runs; conditional formatting shows the highest values (red) and grades down through white to the lowest values (green) (for each measurement location) or blue (for the average) .......................................................... 72
Table 3-6: Vein stiffness properties tested in elastic model trial runs .................................................. 73
Table 3-7: Standard deviation analysis of normalized total displacements in elastic vein model trial runs; conditional formatting shows the highest values (red) and grades down through white to the lowest values (green) (for each measurement location) or blue (for the average) .......................................................... 75
Table 3-8: Standard deviation analysis of normalized total displacements in elastic rockmass model comparison; conditional formatting shows the highest values (red) and grades down through white to the lowest values (green) (for each measurement location) or blue (for the average) .......................................................... 76
Table 3-9: Joint strength properties tested in plastic model trial runs ................................................. 77
Table 3-10: Vein strength properties tested in plastic model trial runs ............................................... 78
Table 3-11: Standard deviation analysis of normalized yield depth in plastic joint model trial runs; conditional formatting shows the highest values (red) and grades down through white to the lowest values (green) (for each measurement location) or blue (for the average) .......................................................... 79
Table 3-12: Standard deviation analysis of normalized yield depth in plastic vein model trial runs; conditional formatting shows the highest values (red) and grades down through white to the lowest values (green) (for each measurement location) or blue (for the average) .......................................................... 80
Table 3-13: Standard deviation analysis of normalized plastic yield depth in rockmass model comparison; conditional formatting shows the highest values (red) and grades down through white to the lowest values (green).................................................................................................................................81
Table 3-14: Descriptions and corresponding material CGSI values for structure in each model..........84
Table 3-15: Stiffness and strength properties of intact rock for Case A.............................................86
Table 3-16: Stiffness and strength properties of explicit structure in Case A.................................86
Table 3-17: Stiffness and strength properties of intact rock and soil overburden for Case B........88
Table 3-18: Stiffness and strength properties of explicit structure in Case B.................................89
Table 3-19: GSI properties of observed rockmass structure...............................................................92
Table 3-20: Intact properties of hydrothermal mafic complex .........................................................97
Table 3-21: Geometry of modelled rockmass structure .....................................................................98
Table 3-22: Mechanical properties of rockmass structure ...............................................................98
Table 3-23: Hoek-Brown properties of hydrothermal andesite sorted by laboratory test failure mode....106
Table 4-1: Guidelines for analyzing rock failure as shearing or spalling based on rock strength ratio (compressive / tensile) (Diederichs 2014, 2007) .........................................................................................................................120
Table 4-2: In situ stresses for studied section of New Mine Level undercut level .........................127
Table 4-3: UCS values from laboratory test data for each lithology used in range for predicted depths of spalling........................................................................................................................................145
Table 4-4: In situ stress magnitudes and orientations for case drifts A and B in the El Teniente New Mine Level ........................................................................................................................................145
Table 4-5: Calculated means and ranges of maximum tunnel stress / UCS for each lithology .........147
Table 4-6: Normalized depths of overbreak (r/a) for each lithology with drift radii (a) of 2.2 m ......147
Table 4-7: Maximum and minimum predictive functions for heterogeneous complex rockmasses based on observed brittle overbreak at the El Teniente New Mine Level Project .........................................................151
Table 5-1: Modal mineralogies of the vein samples ..........................................................................165
Table 5-2: Behaviour characteristics of the vein minerals ...............................................................166
Table 5-3: Elastic properties of vein minerals ....................................................................................170
Table 5-4: Strength properties of vein minerals ................................................................................172
Table 5-5: Calibrated vein stiffness properties. The highlighted rows are used to easily distinguish between the different intact Young’s Modulus (E_i) cases..............................................................174
Table 5-6: Calibrated Mohr-Coulomb vein strength properties. The highlighted rows are used to easily distinguish between the different intact Young’s Modulus (E_i) cases ..........................................................174
Table 5-7: Quantitative numerical results of the amount of vein yield the tunnel models .............180
Table 6-1: Geometry properties of vein sets ....................................................................................195

xxxii
## List of Abbreviations

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$a$</td>
<td>Hoek-Brown strength criterion material constant</td>
</tr>
<tr>
<td>$c$</td>
<td>Mohr-Coulomb cohesion</td>
</tr>
<tr>
<td>CGSI</td>
<td>Composite Geological Strength Index</td>
</tr>
<tr>
<td>CI</td>
<td>Crack Initiation</td>
</tr>
<tr>
<td>$D$</td>
<td>Generalized Hoek-Brown strength criterion damage factor</td>
</tr>
<tr>
<td>DEM</td>
<td>Discrete Element Method</td>
</tr>
<tr>
<td>DISL</td>
<td>Damage Initiation Spalling Limit</td>
</tr>
<tr>
<td>DGR</td>
<td>Deep Geological Repository</td>
</tr>
<tr>
<td>$E, E_i$</td>
<td>Intact Young’s Modulus</td>
</tr>
<tr>
<td>EDZ</td>
<td>Excavation Damage Zone</td>
</tr>
<tr>
<td>$EDZ_i$</td>
<td>Inner Excavation Damage Zone</td>
</tr>
<tr>
<td>$EDZ_o$</td>
<td>Outer Excavation Damage Zone</td>
</tr>
<tr>
<td>EIZ</td>
<td>Excavation Influence Zone</td>
</tr>
<tr>
<td>FEM</td>
<td>Finite Element Method</td>
</tr>
<tr>
<td>GSI</td>
<td>Geological Strength Index</td>
</tr>
<tr>
<td>H-B</td>
<td>Hoek-Brown strength criterion</td>
</tr>
<tr>
<td>HDZ</td>
<td>Highly Damaged Zone</td>
</tr>
<tr>
<td>ISRM</td>
<td>International Society for Rock Mechanics</td>
</tr>
<tr>
<td>$J_a$</td>
<td>Joint alteration number</td>
</tr>
<tr>
<td>$J_r$</td>
<td>Joint roughness number</td>
</tr>
<tr>
<td>$J_{\text{Cond}}$</td>
<td>Joint Condition Rating component in RMR$_{89}$</td>
</tr>
<tr>
<td>JCS</td>
<td>Joint Compressive Strength</td>
</tr>
<tr>
<td>JRC</td>
<td>Joint Roughness Coefficient</td>
</tr>
<tr>
<td>$K$</td>
<td>Ratio of horizontal to vertical in situ stresses</td>
</tr>
<tr>
<td>$K_n$</td>
<td>Normal stiffness</td>
</tr>
<tr>
<td>$K_s$</td>
<td>Shear stiffness</td>
</tr>
<tr>
<td>M-C</td>
<td>Mohr-Coulomb strength criterion</td>
</tr>
<tr>
<td>$m$</td>
<td>Hoek-Brown strength criterion material constant</td>
</tr>
<tr>
<td>$m_b$</td>
<td>Rockmass Generalized Hoek-Brown strength criterion material constant</td>
</tr>
<tr>
<td>$m_i$</td>
<td>Intact Generalized Hoek-Brown strength criterion material constant</td>
</tr>
<tr>
<td>L&amp;ILW</td>
<td>Low and Intermediate Level Waste</td>
</tr>
</tbody>
</table>
MLA  Mineral Liberation Analysis
NWMO  Nuclear Waste Management Organization
OPG  Ontario Power Generation
Q  Norwegian Geotechnical Classification System
Q'  Modified Q system
RMR  Rock Mass Rating classification system
RMR$_{76}$  1976 version of RMR
RMR$_{89}$  1989 version of RMR
RQD  Rock Quality Designation index
SEM  Scanning Electron Microscope
UCS  Uniaxial (or Unconfined) Compressive Strength
UDEC  Universal Distinct Element Code
XRD  X-Ray Diffraction
s  Hoek-Brown strength criterion material constant
$\delta_n$  Normal displacement
$\delta_s$  Shear displacement
$\phi$  Mohr-Coulomb friction angle
$\sigma_c$, $\sigma_{ci}$  Uniaxial or unconfined compressive strength
$\sigma_H$  Maximum horizontal in situ stress
$\sigma_h$  Minimum horizontal in situ stress
$\sigma_{\text{max}}$  Maximum tangential boundary stress
$\sigma_n$  Normal stress
$\sigma_t$  Tensile strength
$\sigma_v$  Vertical in situ stress
$\sigma_1$  Maximum principal stress
$\sigma'_1$  Maximum effective principal stress
$\sigma_2$  Intermediate principal stress
$\sigma_3$  Minimum principal stress
$\sigma'_3$  Minimum effective principal stress
$\tau$  Shear stress
$\nu$  Poisson’s ratio
Chapter 1

Introduction

1.1 Purpose of Study

Modern civil and mining engineering excavations are increasingly being constructed in complex rockmasses and situated at deeper horizons that are subject to high in situ stresses. Examples of modern civil excavations include base tunnels for irrigation and vehicle transportation, as well as deep geological repositories for the permanent storage of nuclear waste (see Figure 1-1). Examples of modern mining excavations include giant block cave mines for low grade disseminated orebodies such as porphyry deposits, which require hundreds of kilometers of excavation infrastructure and can be situated more than 1 km below ground surface (see Figure 1-1).

Conventional rockmasses are comprised of micro-scale intact rock blocks that are bounded by macro-scale fractures. Macro-scale fractures such as joints, bedding, foliations, and other discontinuities are termed interblock structure in this thesis. Complex rockmasses also include meso-scale healed structure that behaves as part of the intact rock in situ. These meso-scale healed structures such as hydrothermal veins, veinlets, and stockwork, lithified interbed nodular features in sedimentary rocks, and others, are termed intrablock structure in this thesis (examples shown in Figure 1-2). The mechanical behaviour of intrablock structure is primarily controlled by infill mineralogy and geometrical properties such as thickness, persistence, and orientation. Intrablock structure can behave as part of the “intact” rock in high-quality, undisturbed drill core, but influences rockmass shear and tensile strength at a larger scale and can control fragmentation after moderate disturbance and comminution.
Figure 1-1: Examples of modern excavations in complex rockmasses where intrablock structure can influence rockmass behaviour; (top) Olmos trans-Andean water transport tunnel (Diederichs et al., 2013); (middle) Ontario’s deep geological repository for low to intermediate level nuclear waste storage (courtesy of NWMO); (bottom) 3-Dimensional model of the El Teniente block cave mine showing undercut levels with hundreds of kilometers of excavation infrastructure (modified after Pardo et al., 2012)
Figure 1-2: Examples of intrablock structure; (a) drill core with hydrothermal pink gypsum and white quartz veins; (b) drill core with hydrothermal quartz veins with variable thicknesses from 0.5 mm to 7 cm; (c) hydrothermal quartz veins at an excavation face (rock bolts and plates for scale); (d) sample cube of Cobourg limestone with intrablock structure defined as tortuous clay-rich layers between calcite-rich nodules (cube dimensions are 40 cm); (e) unrolled scan of cylindrical core surface of Cobourg limestone showing more detail of intrablock structure (dark grey layers)

While intact rock and interblock structures are routinely considered in geotechnical engineering design, intrablock structure was considered to be irrelevant to rockmass behaviour and stability in early to mid-20th century shallow excavation design. However, field observations in increasingly deep modern
excavations, that are exposed to higher and more complex stress paths, have demonstrated that intrablock structure can have a significant influence on rockmass behaviour and should, therefore, be included in rockmass characterization for geotechnical design.

Conventional design practices are typically not designed to consider the effect of intrablock structure. The purpose of this research is to address these issues by developing new methodologies and tools to incorporate intrablock structure into rockmass characterization and geotechnical design practice. In particular, the research outcomes range from field characterization and failure prediction techniques to continuum and discontinuum numerical modelling. The author is confident that this research provides valuable contributions to the evaluation of complex rockmasses for the effective geotechnical design of modern excavations.

1.2 Empirical Geotechnical Design Practices

The majority of routine geotechnical design follows rockmass classification methodologies that have been empirically correlated to observed excavation behavior to develop design charts that prescribe primary ground support. The Rock Mass Rating (RMR) (Bieniawski, 1973, 1989), Modified Rock Mass Rating (MRMR) (Laubscher, 1977; Laubscher, 1990; Laubscher and Jakubec, 2001), and Norwegian Tunnelling Index, Q (Barton et al., 1974), remain some of the most popular rockmass classification systems around the world. These tools resulted in a significant improvement to reliable geotechnical design when faced with impractical field scale testing to assess rock strength parameters.

RMR and Q are based on numerous tunnel and mine cases. MRMR is an extension of the RMR system for specific application to mining projects. Most of the case histories on which it is based are from block caving operations. The routine use of these classification systems for a wide variety of projects continues to provide additional case studies and therefore continuing opportunity to refine parameter calibration. Especially for long term projects that have already been operating for years or decades, consistent application of a classification system enables observational design and updates of excavation methods and support systems throughout the project. Nonetheless, problems arise when changes in
geology, stress, and other conditions occur along an excavation advance. Even for conventional rockmasses, the limited input parameters in classification systems result in output data that does not adequately capture the full impact of rockmass behaviour on excavation stability (van der Pouw Kraan, 2014).

These empirical rockmass classification systems include little to no consideration of intrablock structure. The RMR system does not consider intrablock structure at all. The joint condition rating section of RMR ranges from “very rough surfaces, not continuous, no separation, unweathered wall rock” to “soft gouge > 5 mm thick or separation > 5 mm, continuous” (Bieniawski, 1989). The Q system has a provision for “tightly healed” joint alteration which arithmetically improves the joint alteration rating (and overall Q value) from a joint with “surface staining only” by 33% (Barton et al., 1974). There is no allowance for the wide range of strengths found in intrablock structures that are primarily controlled by various healed infill mineralogies. Furthermore, rockmass structure is averaged out in a single Q value, which eliminates the ability to capture anisotropic behaviour created by structures. The MRMR system (Laubscher and Jakubec, 2001) has a vein adjustment factor that reduces the strength of the intact rock block by means of assessment of the vein frequency (veins per meter) and infill hardness. This factor does not influence the joint condition rating. The Mohs hardness number is used as an analogue to describe the strength of veins (Mohs, 1825). The range of the Mohs hardness scale applied to MRMR is only 0.2 to 5 because values greater than 5 (such as apatite and quartz) were regarded as insignificant by Laubscher and Jakubec (2001). Open fractures are assigned a factor of 1, and the veins in MRMR are only able to weaken the host rock. The procedure developed by Laubscher and Jakubec (2001) involves multiplying the inverse of the Mohs hardness value (shown in Table 1-1) by the vein frequency per meter, to arrive at a fraction of the initial intact rock block strength (see Figure 1-3). Although MRMR is the best empirical classification system for considering veins, it does not account for intrablock structure that strengthens the rockmass and the geometrical and strength parameters are limited. Furthermore, MRMR shares the most significant limitation with RMR and Q, where the final result is a single rank value that does not account for
anisotropy in rockmass structure. Therefore, these rockmass classification systems fundamentally cannot capture anisotropic rockmass behaviour.

**Table 1-1: Inverse values of Mohs hardness for use in MRMR block strength to account for weakening of the rockmass by veins (Laubscher and Jakubec, 2001)**

<table>
<thead>
<tr>
<th>Mohs Hardness number</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mineral examples</td>
<td>Talc, Molybdenite</td>
<td>Gypsum, Chlorite</td>
<td>Calcite, Anhydrite</td>
<td>Fluorite, Chalcopyrite</td>
<td>Apatite</td>
</tr>
<tr>
<td>Inverse Hardness</td>
<td>1.0</td>
<td>0.5</td>
<td>0.33</td>
<td>0.25</td>
<td>0.2</td>
</tr>
</tbody>
</table>

**Figure 1-3: Adjustment factor for MRMR block strength to account for veins within the rock block (modified after Read and Stacey, 2009; Laubscher and Jakubec, 2001)**

**1.3 Numerical Geotechnical Design Practices**

The advancement of numerical modelling in geomechanics has driven significant research and development of numerical design software and modelling procedures. These powerful tools have reduced some reliance on analytical and empirical design solutions, in favour of techniques that are customizable for individual projects with complex geological conditions, excavation geometries, and associated stress
conditions. Numerical models are able to capture all components of a rockmass individually, which preserves the anisotropy of the material. A numerical approach therefore has the potential to be the most accurate representation of a rockmass for geotechnical design. A technological limitation of numerical models is computational capacity, where models of rockmasses with multiple suites of structure have a scale restriction. An engineering limitation of numerical design today in complex rockmasses is the limited detail of geotechnical information that is available for data input. Many site investigation programs are designed to collect geotechnical data through the lens of empirical rockmass classification system parameters. This generally provides enough data for input into continuum numerical models, but not enough for models with explicit or discrete rockmass structure. For continual improvement of geotechnical design for modern projects, numerical methods with explicit rockmass structure are needed in engineering practice to accurately capture rockmass behaviour in order to develop effective and efficient primary ground support systems. For complex rockmasses, numerical models can be implemented to consider multiple suites of rockmass structure that have a variety of geometries and strengths. This is currently the best option for a state-of-the-art approach to incorporate intrablock structures and interblock structures into detailed geotechnical design. Detailed geotechnical design approaches are necessary for projects such as deep geological repositories (DGRs) for nuclear waste storage and giant block caving mine operations. Current designs of DGRs must be able to predict the behaviour of this type of structure under different loading conditions for a design life of one million years. This requires a high level of accuracy in characterizing the behaviour of the host rockmass. In block caving operations, their success relies on controlled fragmentation of the orebody. In hydrothermal porphyry deposits with stockwork vein networks, the veins or vein-wall rock contacts are significant controls on the orientations of fracture development. It is therefore imperative to understand the character of the rockmass structure for an economically successful and safe mining operation.
1.4 Research Objectives

This thesis aims to provide insight into the behaviour of complex rockmasses with intrablock structure and, ultimately, to provide improved engineering tools to characterize intrablock structure for more effective consideration and implementation in modern geotechnical design. The specific research objectives to achieve this are described as follows and summarized in Figure 1-4.

- To critically review the existing understanding of intrablock structure from a geological perspective and the existing consideration of intrablock structure from a geomechanics perspective;
- To develop field characterization tools for complex rockmasses with various combinations of interblock and intrablock structures;
- To investigate the behaviour of intrablock structures in the field and assess their influence on excavation behaviour and stability;
- To develop brittle overbreak estimation tools for excavations in complex rockmasses;
- To develop detailed core logging procedures that improve data capture of interblock and intrablock rockmass structure for use as input to numerical models with explicit rockmass structure;
- To examine the mechanical components of intrablock structure and their variability in thin section to calibrate input parameters for explicit numerical models;
- To delineate methodologies for the inclusion of intrablock structure in numerical models where rockmass structure is included implicitly (continuous models) and explicitly or discretely (discontinuous models);
- To modify existing geomechanical laboratory testing and field rockmass characterization assessment methodologies for application to intrablock structure and complex rockmasses;
- To develop protocols and conduct direct shear laboratory testing of intrablock structure to determine stiffness and strength properties at the laboratory scale for use as inputs to numerical models with explicit or discrete rockmass structure; and
- To investigate the mechanical behaviour in direct shear of interblock and intrablock structure in the Cobourg limestone from the Bowmanville quarry and at the Bruce Deep Geological Repository site and draw correlations to changes in its mineralogical composition.

Figure 1-4: Summary diagram describing the thesis research activities and objectives herein to geotechnical design of complex rockmasses

1.5 Thesis Outline and Contributions

This thesis has been prepared in accordance with the requirements outlined by the School of Graduate Studies at Queen’s University, Kingston, Ontario, Canada. The structure of this thesis follows the manuscript style where the majority of the body chapters has either been published or will be submitted to
peer-reviewed academic journal publications. This thesis consists of nine chapters, which are outlined below. References are presented at the end of each chapter.

In Chapter 1 the importance of rockmass characterization for effective geotechnical design and the increasing significance of healed (intrablock) rockmass structure on rockmass behaviour with depth are briefly discussed.

In Chapter 2 the Hoek-Brown strength criterion and its associated rockmass characterization method, the Geological Strength Index (GSI), is reviewed. A discussion of Finite Element numerical modelling and how rockmass structure is treated in this technique is also included.

In Chapter 3 a new field rockmass characterization tool, the Composite Geological Strength Index (CGSI), is presented, for characterization of rockmasses that contain multiple suites of fractured (intrablock) rockmass structure, healed (intrablock) rockmass structure, or both. The CGSI technique is ultimately useful for providing input parameters to equivalent continuum materials in numerical models with implicit rockmass structure. A numerical investigation is used to compare conventional application of GSI to the new CGSI approach for a complex rockmass. A stepwise optimization technique for implementation of explicit and implicit rockmass structure in numerical models is also presented for rockmasses with multiple distinct suites of structure. The CGSI method is applied to model the implicit structure in this technique. Finally, field observations of a case study of a ventilation adit at the El Teniente mine in Chile are used to validate the CGSI method. Strategies to incorporate intact laboratory test results of failure through intrablock structure into equivalent continuum numerical models are also presented in this chapter. A case study of laboratory test results is used to show that strength envelopes of samples with failure through intrablock structure can be defined using modified parameters from the Hoek-Brown strength criterion, when compared to conventionally accepted strength properties that describe specimen failure through the rock matrix.

In Chapter 4 the application of excavation measurements from a deep drift at the El Teniente mine in Chile is used to develop a brittle overbreak (spalling) prediction tool for heterogeneous, complex
rockmasses, on a mechanistic basis. An in-depth review of empirical and mechanistic based prediction tools for homogeneous rocks is discussed. The mechanistic separation of depths of spalling between different layers of excavation damage zones previously studied for homogeneous rockmasses is correlated to a variety of heterogeneous, complex rockmasses. This investigation results in a prediction tool for depth of brittle failure around an excavation in complex rockmasses.

In Chapter 5 the mechanical stiffness and strength properties of hydrothermal intrablock structure using petrographic thin section analyses coupled with Finite Element numerical simulations of uniaxial compressive strength laboratory tests is examined. The mineralogical composition and grain-scale geometry of the veins and the corresponding mechanical properties of the grains are simulated to determine the compound axial stress-strain behaviour of the vein. Normal stiffness, shear stiffness, and Mohr-Coulomb criterion strength properties are calibrated to explicit structural elements for a variety of vein types. This is an essential step toward finding accurate mechanical properties of intrablock structure that are required for explicit numerical models.

In Chapter 6 a new extension of the Mohr-Coulomb strength criterion for healed (intrablock) rockmass structure is presented. This three-stage strength criterion considers primary “peak” strength to be controlled by the strength of the healed infill material; secondary post-peak “residual” strength occurs immediately after fracturing of the vein and resembles a clean joint before sliding; tertiary “ultimate” strength develops after subsequent shear displacement of the structure and resembles the residual state of a conventional fracture. This three-stage Mohr-Coulomb criterion is developed using overbreak data from the El Teniente mine in Chile and is applicable to defining mechanical properties of explicit rockmass structure for numerical models.

In Chapter 7 new progressively detailed core logging methods for data collection of complex rockmasses with healed (intrablock) structure are tested and compared to existing core logging methods. These core logging methods were developed at the Esperanza and Caracoles mines in northern Chile as well as the Nickel Rim South mine in Sudbury, Ontario, Canada. The amount of detail collected in each
method is applied to excavation scale numerical models to investigate the influence of the data on simulated rockmass behaviour. These methods are also applied to drill core characterization of the Cobourg limestone. The excavation scale cases address complex rockmasses in hydrothermal and carbonate sedimentary geological environments. When compared to the relative core logging times and cost of each method, suggestions for the optimal balance of detail gained versus expense for different types of geotechnical projects are discussed. The detailed data collection for complex rockmasses, which includes intrablock structure, is intended for use as input to detailed numerical models with explicit rockmass structure.

In Chapter 8 new laboratory direct shear testing protocols for intrablock structure are presented. Laboratory testing of fractures and intrablock structure in the Cobourg limestone was conducted to evaluate their normal stiffness, shear stiffness, shear strength, and post-peak dilation properties for use as input parameters in numerical models with explicit or discrete structure. The Cobourg limestone used in this testing program was sourced from the St. Marys Cement Bowmanville quarry near Bowmanville, Ontario, Canada, which is a stratigraphic equivalent to the Cobourg limestone that occurs approximately 700 m below ground surface at the Bruce deep geological repository (DGR) site near Kincardine, Ontario. A comparison of these laboratory results is made to direct shear test data of samples from the DGR site to examine changes in mechanical properties between sample source locations and evaluate the suitability of the Bowmanville quarry Cobourg limestone as an analogue for anticipated mechanical behaviour at the repository depth. An investigation of the mineralogical composition of these rocks was also conducted using X-Ray Diffraction and Mineral Liberation Analysis to correlate mineralogy changes between source locations to changes of mechanical properties in direct shear.

In Chapter 9 a discussion and conclusions of the key findings of this thesis, recommendations for future research, and a summary of contributions made through this research are presented.
1.6 References


This page left blank
Chapter 2

Numerical Methods, the Hoek-Brown Strength Criterion, and the Geological Strength Index

2.1 Rockmass Classification

Rockmass data can be collected from combinations of field observations from surface outcrops, drill core, and pilot tunnels, as well as laboratory testing for mechanical properties. This rockmass characterization process requires an assessment of the micro-scale intact rock strength and macro-scale rockmass structure to evaluate and quantify mechanical behaviour of the rockmass in terms of stiffness and strength parameters. Preliminary assessments of rockmass properties can be made in the field using descriptive observations and correlations to empirical systems that quantify these observations. More detailed assessments are conducted through laboratory tests. Standard laboratory tests for intact rock strength include Uniaxial Compressive Strength (UCS), axisymmetric triaxial strength, and tensile strength tests. Micro-scale mineral grain interaction controls intact rock strength and behaviour. In many cases, macro-scale rockmass structures such as joints, bedding, and foliation discontinuities continue to be evaluated primarily through the lens of field observation-based empirical rockmass classification systems, such as the Rock Mass Rating system (RMR) (Bieniawski 1973, 1989), the Mining Rock Mass Rating system (MRMR) (Laubscher 1990; Laubscher and Jakubek 2001), and the Tunnelling Quality Index (Q) (Barton et al. 1974). These tools were introduced at a time when no practical numerical tools were available; geotechnical design relied on an empirical process. These classification systems tend to become less relevant as both civil and mining underground excavations go deeper encountering higher in situ stress conditions and encounter less routine geohazards, or as mining methods such as block caving are adapted to unconventionally large, deep, and geomechanically competent orebodies.

After decades of relying on empirical classification systems to assess rockmass quality, a characterization system that is dependent on direct geological field observations was created: the
Geological Strength Index (GSI) (Hoek, 1994; Hoek et al., 1995). GSI has evolved to be used in conjunction with the Generalized Hoek-Brown rock and rockmass strength criterion by degrading the failure envelope from intact rock to rockmass strength, where the parameter values can be directly used as numerical model inputs (Hoek and Brown 1997; Hoek et al. 2002).

Conventional characterization practices are tailored toward the use of one or more common empirical classification systems like the Rock Quality Designation (RQD) (Deere et al. 1969), RMR, Q, and MRMR, which all consider properties of interblock structure by some means. The selection of a classification system differs between projects and groups responsible for collecting and using the data. The use of RQD has evolved to be a component of RMR, MRMR, and Q rather than being applied independently. RMR, Q, and MRMR are the most commonly used classification systems in underground mine settings because of convention and ease of use. RMR and Q were designed for underground support design and have been calibrated to numerous tunnel and mine cases. RMR is comprised of weighted factors for UCS of intact rock, RQD, spacing and condition of discontinuities, groundwater conditions, and orientation of discontinuities relative to the excavation. Q quantitatively describes rockmasses based on in situ rock quality, joint condition, and stress state; the output is directly applied to stability and support recommendation design charts. MRMR extends the in situ rockmass quality assessment of RMR for application in mining projects by including the effects of mining activities on the rockmass. These include factors for weathering, joint orientation, mining-induced stresses, blasting, and water and ice. Most of the case histories on which it is based are from caving operations.

2.1.1 Difficulties with Multiple Structural Elements

Since conventional classification systems were developed as an empirical basis for support design, a certain amount of conservatism is essential to account for the extreme variability inherent in geological and structural settings. The average to poor rockmass structure characteristics are typically selected to represent an entire rockmass to maintain conservatism. Furthermore, properties from multiple joint sets are combined, so the strength of the rockmass becomes amalgamated into an average value. This practice
becomes especially problematic when there is a significant difference in discontinuity strength between sets, which makes it difficult to recognize which failure modes are likely to dominate a system (and when). The weakest discontinuity in a rockmass will be the limiting factor of local behaviour and will control the failure mode. These practices lead to conservative rockmass strength, resulting in overdesign of excavation support requirements.

The advances in numerical modelling present an opportunity for more detailed design that separates rockmass structure into components of similar mechanical (stiffness and strength) properties. This is advantageous for modern geotechnical design since it can reduce the conservatism found in traditional empirical approaches. It is simultaneously important to develop a sound understanding of the mechanisms involved with structural behaviour when implementing advanced numerical tools instead of relying on input properties determined using conservative assumptions. The Composite GSI approach presented in Chapter 3 of this thesis offers a solution that addresses strength variability in sets of interblock structure in addition to strength variability between suites of interblock and intrablock structures.

2.2 Numerical Methods

Numerical methods are very powerful tools that can be used to solve more diverse and complex geomechanics problems than analytical or empirical methods, which rely on closed form solutions or are limited to previously documented problem definitions in certain conditions. Numerical methods in geomechanics range between purely continuum and discontinuum approaches (Figure 2-1). Purely continuum methods (e.g. Boundary Element Methods, BEM) represent the problem using homogenous materials that implicitly incorporate rockmass structure through the stiffness properties and strength criterion of the material (e.g. Crouch and Starfield, 1983). Purely discontinuum methods (e.g. Particle Flow Code, PFC) discretely model all discontinuities and boundaries within the problem (e.g. Itasca, 2008). Several intermediate methods can model rockmass structure both implicitly and explicitly (or discretely) and are currently popular tools at the excavation scale, such as Finite Element Method (FEM)
(e.g. Phase² or RS2 by RocScience, 2012, 2015) and Discrete Element Method (DEM) (e.g. UDEC by Itasca, 2013). Greater amounts of explicit or discrete elements are more computationally demanding; therefore, there are scale limitations for more explicit and discrete numerical models with present day computation capacity. FEM numerical modelling is used to carry out the research in this thesis because of its capability to model implicit and explicit rockmass structure at both the laboratory testing and underground excavation scales. At the excavation scale, multiple suites of rockmass structure are analyzed in a single model. The Phase² FEM code by RocScience (2012; updated and rebranded as RS2 in 2015) is used for this study.

Figure 2-1: Range of numerical methods used in geomechanics from continuum to discontinuum codes

2.2.1 Finite Element Method (FEM)

The Finite Element Method (FEM) is one of the most popular numerical methods in engineering (e.g. Zienkiewicz and Taylor, 2000) and geomechanics (e.g. Puzrin, 2012; Bobet, 2010; Pande et al., 1990; Wittke 1990) because it can accommodate stress anisotropy, material heterogeneity, complex boundary conditions and various constitutive relationships (Jing, 2003). The premise of FEM is the discretization of the model domain into a mesh of a finite number of contiguous sub-domains that are defined by a fixed number of shared nodes (Jing, 2003). Therefore, FEM is fundamentally a continuum approach that relies on principles of infinitesimal strain, and the mesh elements cannot detach from each other. However,
explicit structural elements can simulate separation of the surrounding continuum material by a reduction in normal stiffness after yield (RocScience, 2015). While displacements measured in a FEM model of discontinuous media may not accurately reflect real-world magnitudes since they are limited to small strain computation, they remain useful for semi-quantitative comparisons between models. In cases of continuous media such as soil or extremely large scale models of rockmasses, the displacement magnitudes in FEM models are acceptable reflections of the real world.

In a broad sense, solutions for stress analysis problems in a continuum material are governed by partial differential equations. The FEM approach discretizes the complex material and boundary conditions found in geomechanics problems into simple calculable geometrical elements. The solution is then approximated by iteratively solving algebraic equations for each element and reassembling the components to form the solution for the domain. A basic assumption of FEM is that the unknown displacement function over each element can be approximated by a function of its nodal displacement values in polynomial form (Jing 2003). The major steps to calculate stresses using FEM are as follows (Puzrin 2012; Jing 2003; Pande et al. 1990; Wittke 1990):

a) *Discretize domain into simple elements*

The domain is discretized into sub-domains of elements such as triangles or rectangles in 2-Dimensions (or tetrahedrons and rectangular parallelepipeds in 3-Dimensions). Each element has a specified number of nodes on corners that are shared between adjacent elements. Higher order elements have nodes on both corners and edges. The displacements at these nodes are the main unknowns of the problem.

The regularity of each element shape creates a high-quality mesh; elongated elements with small angles will degrade the quality of the solution. The total amount of elements in a domain will control the accuracy and computation time. The size of elements can vary in a domain where small elements (i.e. a denser mesh) are desired in areas where larger stress changes are expected, such as close to an excavation.
boundary. Mesh generation is very flexible and can easily accommodate complex excavation and external boundary geometries, as well as anisotropic and heterogeneous networks of explicit structural elements.

b) Derive element stiffness matrix

Displacements are calculated at the nodes of each element. Shape functions \( (N_i) \) are used to derive the interpolated displacement within each element from the nodes. Shape functions vary depending on the element geometry to discretize the domain; an example of shape functions for a triangular element is shown in Figure 2-2. The problem can now be represented, using the shape functions, by an algebraic system of equations written as follows:

\[
[K^e][u^e] = \{f^e\}
\]

where \([K^e]\) is the element stiffness matrix, \(\{u^e\}\) is the nodal value vector of the unknown variables (displacements), and \(\{f^e\}\) is the force vector composed of body forces and initial/boundary conditions. The element stiffness matrix \([K^e]\) consists of the element geometry matrix, \([B]\), by derivatives of the shape functions, and the elasticity constants for the material, matrix \([D]\):

\[
[K^e] = \int [B^T][D][B] \, dv
\]

\[(2.2)\]

![Shape Functions](image)

**Figure 2-2: FEM shape function for a six-noded triangular mesh element (after Pande et al., 1990)**

c) Assemble global stiffness matrix

The stiffness of each element is ‘added’ to form a global stiffness matrix for the entire domain. The size of the global stiffness matrix corresponds to the total number of degrees of freedom in the mesh (e.g. twice the number of nodes for 2D problems).


d) Define boundary conditions

Available boundary conditions include prescribed displacements or forces along specified sections of the external boundary. To model a single deep tunnel, for instance, external boundaries are typically defined to have zero displacements and must be located outside of the excavation-induced stress field. Alternatively, to model a series of parallel tunnels at consistent spacing, the zero displacement external boundaries are located at half of the tunnel spacing to act as an axis of symmetry.

e) Solve global equations to calculate displacements, strains, and stresses

Equilibrium equations for each node sum the nodal forces to assemble element stiffness matrices into the global stiffness matrix, \([K]\), for the entire continuum. A solution of the resulting reduced system of linear algebraic equations produces the vector of nodal displacements, \(\{u_n\}\).

Constitutive relationships are applied to the strains to define the stresses in each element. For rockmasses, the following stiffness relationship is applied:

\[
\{\sigma\} = \{\varepsilon\}[D]
\]  \hspace{1cm} (2.3)

The material elasticity constants in \([D]\) are composed of Young’s Modulus and Poisson’s Ratio.

2.2.2 Explicit Structural Elements

Rock mechanics motivated the creation of fracture elements in FEM. Several fracture element types have been developed for 2D and 3D problems (e.g. Goodman et al., 1968, Ghaboussi et al., 1973, Zienkiewicz et al., 1970, Buczkowski and Kleber, 1997, among others). The FEM software Phase\(^2\) by RocScience (2015) uses the Goodman joint element (Goodman et al., 1968). The Goodman joint element has nodes along the joint connecting to mesh nodes on either side of the element (see Figure 2-3). However, the joint element has zero theoretical thickness, and its behaviour is controlled by defined normal and shear stiffness properties \((K_n \text{ and } K_s)\). Also, failure criteria such as Mohr-Coulomb or Barton-Bandis (Barton and Bandis, 1990) can be used to consider peak and post-peak strength behaviour. Shear deformation is controlled by the assigned stiffness and strength properties while a reduction in stiffness simulates large-scale openings and complete detachment of the joint element post-peak.
Present computation capacity limits the maximum relative density of joint elements with respect to the spacing of external boundaries. In the opinion of the author, it is currently practical to create 2D models with multiple intersecting joint networks with an average spacing at 0.3% of the external boundary.

![Diagram of Goodman joint fracture element for FEM models](Goodman joint fracture element for FEM models (Goodman et al., 1968))

2.2.3 Networks of Structural Elements

Explicit structural (joint) elements can be generated in networks with defined deterministic or statistical geometric properties. Parallel statistical and Voronoi polygonal joint networks have been implemented in Phase² (RS2) 2D FEM models for this research. The input parameters for the parallel statistical and Voronoi joint network models are described in Table 2-1.

Multiple parallel joint networks can be used to create joint sets of different orientations and other geometries; mechanical properties can be uniquely defined for each set. Voronoi joint networks are created by a 2D tessellation that randomly subdivides the domain into non-overlapping convex polygon cells. The Poisson point process begins the Voronoi generation by seeding the domain with points that are randomly distributed. The Voronoi cell corresponding to each seed point is the planar region closer to that seed than any other. The bounding segments of each cell are equidistant lines between pairs of neighbouring seed points (RocScience, 2015). The polygon growth process must also have a constant growth rate from each seed point (Dershowitz and Einstein, 1988). The Voronoi tessellation technique has been widely accepted for modelling micro-scale rock structure, such as mineral grain boundaries (e.g. Lan et al., 2010).
Table 2-1: Geometrical parameters for selected joint network models in Phase² (RS2) (after RocScience, 2015)

<table>
<thead>
<tr>
<th>Input Parameter</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Parallel Statistical</strong></td>
<td></td>
</tr>
<tr>
<td>Orientation</td>
<td>For this 2D model, the dip of the trace plane is used. The orientation cannot have a statistical distribution because the model assumption is that the joints are parallel.</td>
</tr>
<tr>
<td>Spacing</td>
<td>The perpendicular distance between parallel joint planes.</td>
</tr>
<tr>
<td>Length</td>
<td>Infinite (joints are continuous across the domain) or discontinuous length.</td>
</tr>
<tr>
<td>Persistence</td>
<td>Applicable for discontinuous length; defined as the ratio of joint length to total length along the joint planes; value is between 0 (zero joint length) to 1 (fully persistent or infinite); simulates rock bridges.</td>
</tr>
<tr>
<td>Joint end condition</td>
<td>Open or closed joint ends; open joint ends have two nodes at the ends; closed joints have one node at the ends which prevents relative movement (sliding or opening) at that end.</td>
</tr>
</tbody>
</table>

**Voronoi**

<table>
<thead>
<tr>
<th>Input Parameter</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Joint density</td>
<td>Controls spacing of Voronoi joint network segments by density (number of Voronoi cells per area) or average joint length of the cell wall segments.</td>
</tr>
<tr>
<td>Regularity</td>
<td>The regularity of polygon shapes is controlled by relative seed locations for polygon growth; ranges from “irregular” to “regular hexagon” patterns (see Figure 2-4).</td>
</tr>
<tr>
<td>Joint end condition</td>
<td>Open or closed joint ends; open joint ends have two nodes at the ends; closed joints have one node at the ends which prevents relative movement (sliding or opening) at that location.</td>
</tr>
</tbody>
</table>

Figure 2-4: Range of Voronoi polygon shape regularity available in Phase² (modified after RocScience, 2015)
2.2.4 Implementation of the Hoek-Brown Strength Criterion and Geological Strength Index

The initial stages of numerical modelling for geotechnical design regularly begin with an equivalent continuum representation of a rockmass, where the peak and post-peak behaviours of discontinuities are represented by a strength criterion. The Generalized Hoek-Brown rock strength criterion is commonly used in rock mechanics, where the failure envelope for intact rock is defined by the unconfined compressive strength (UCS) and the Hoek-Brown material constant, \( m \) (Hoek et al., 2002). The influence of geological structure on the relationship between intact rock strength and rockmass strength is accounted for in the Hoek-Brown criterion by the Geological Strength Index (GSI), which is used to modify the failure envelope of the intact rock to a reduced strength profile (e.g. Hoek et al., 1995; Hoek and Brown, 1997; Hoek and Marinos, 2000; Hoek et al., 2002; Hoek et al., 2013). The partnership of the Hoek-Brown criterion and GSI is particularly useful for numerical modelling because geotechnical field observations of a rockmass are directly incorporated into the strength properties.

2.3 Development of the Hoek-Brown Strength Criterion and Geological Strength Index

The Hoek-Brown strength criterion was originally developed to be a basic rockmass strength criterion suitable for general practical application to estimate intact rock and rockmass strength for underground excavation design (Hoek and Brown, 1980, 1988). The otherwise lack of suitable strength criteria for rockmasses at the time of its creation resulted in widespread use of the Hoek-Brown criterion by geologists and geotechnical engineers. The Hoek-Brown criterion is fundamentally applicable only to isotropic rockmasses where the rockmass behaviour is dominated by interlocking blocks, shear failure, and rotation of blocks formed by intersecting structural features (e.g. Hoek and Brown, 1997). In the context of numerical modelling with current computation abilities, sparse anisotropic rockmass features should be modeled explicitly while isotropic rockmass features are modelled implicitly as an equivalent continuum material.

The empirical criterion is based on analyses of experimental data for intact rock and rock discontinuities that show the relationships between major and minor principal stresses and between shear
and normal stresses at failure are nonlinear (Hoek and Brown 1980). Hoek and Brown (1980) applied the concept of the original Griffith theory for tensile effective normal stresses (Griffith, 1924) to a trial and error process to create the first form of the criterion:

\[
\frac{\sigma_1}{\sigma_c} = \frac{\sigma_3}{\sigma_c} + \sqrt{m \frac{\sigma_3}{\sigma_c} + s}
\]

Where \(\sigma_1\) is the major principal stress at failure; \(\sigma_3\) is the minor principal stress at failure, or in the case of a conventional triaxial test, the confining pressure; \(\sigma_c\) is the uniaxial compressive strength of the intact rock material; \(m\) and \(s\) are empirical constants that depend on the rock properties. \(\sigma_c\) is used because it is one of the most widely quoted constants in rock mechanics and has the highest likelihood of being available compared to other rock strength data. Curve fits of the Hoek-Brown criterion to data in five categories of rock types (including intact rock and various qualities of rockmasses) show that \(m\) and \(s\) decrease from their maximum values at the intact rock with increasing joint intensity and weathering. Furthermore, values of \(m\) for intact rock are dependent on rock types while \(s\) describes the rock quality. The constant \(s\) ranges from 1 for intact rock (with nonzero, finite tensile strength) to 0 for jointed rockmasses (zero tensile and cohesive strength when the effective normal stress is zero) (Hoek and Brown, 1980). Although laboratory and field test data were used to apply the Hoek-Brown criterion to the evaluated case studies, it was deemed impractical to expect every geotechnical project to have the budget and patience to conduct individual tests for preliminary strength properties. Therefore, a correlation of \(m\) and \(s\) values was developed considering field observations and assessments of rockmass strength of Panguna Andesite (Hoek and Brown, 1980) using established rockmass classification systems, Q (Barton et al., 1974) and RMR (Bieniawski, 1976) and the following relationship defined by Bieniawski (1978):

\[
RMR = 9 \ln Q + 44
\]

The relationships between \(m\) and \(s\) values with respect to Q and RMR are shown in Figure 2-5. Approximate Hoek-Brown criterion equations and interpolated Q and RMR ratings for the five cases of rock types discussed in Hoek and Brown (1980) are shown in Table 2-2, where the general trends of \(m\)
and $s$ can also be observed. Hoek and Brown (1980) emphasized that the Hoek-Brown criterion relationships shown here in Table 2-2 be “based on very sparse data and are therefore very approximate; they should be used only as rough guides in preliminary design calculations”.

Figure 2-5: Correlations of $m$ and $s$ to rockmass classification systems Q and RMR for Panguna Andesite data (Hoek and Brown, 1980). This relationship was used to approximate $m$ and $s$ values for various rockmasses shown in Table 2-2.
Table 2-2: Approximate equations for principal stress relationships and Mohr envelopes for five categories of intact rock and jointed rockmasses (redrafted from Hoek and Brown, 1980)

<table>
<thead>
<tr>
<th>Rock quality (1)</th>
<th>Carbonate rocks with well developed crystal cleavage (dolomite, limestone and marble) (2)</th>
<th>Lithified argillaceous rocks (mudstone, siltstone, shale and slate nortite and quartz-diorite) (3)</th>
<th>Arenaceous rocks with strong crystals and poorly developed crystal cleavage (sandstone and quartzite) (4)</th>
<th>Fine grained polyminallic igneous crystalline rocks (andesite, dolerite, diabase and rhyolite) (5)</th>
<th>Coarse grained polyminallic igneous and metamorphic crystalline rocks (amphibolite, gabbro, gneiss, granite, nortite and quartz-diorite) (6)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intact rock samples—laboratory size rock specimens free from structural defects (CSIR rating 100+; NGI rating 500)</td>
<td>$\sigma_{ls} = \sigma_{sn} + \sqrt[3]{7\sigma_{sn} + 1.0}$</td>
<td>$\sigma_{ls} = \sigma_{sn} + \sqrt[3]{10\sigma_{sn} + 1.0}$</td>
<td>$\sigma_{ls} = \sigma_{sn} + \sqrt[3]{15\sigma_{sn} + 1.0}$</td>
<td>$\sigma_{ls} = \sigma_{sn} + \sqrt[3]{17\sigma_{sn} + 1.0}$</td>
<td>$\sigma_{ls} = \sigma_{sn} + \sqrt[3]{25\sigma_{sn} + 1.0}$</td>
</tr>
<tr>
<td>$\tau_s = 0.816(\sigma_n + 0.140)_{0.687}$</td>
<td>$\tau_s = 0.918(\sigma_n + 0.099)_{0.677}$</td>
<td>$\tau_s = 1.044(\sigma_n + 0.067)_{0.692}$</td>
<td>$\tau_s = 1.086(\sigma_n + 0.059)_{0.696}$</td>
<td>$\tau_s = 1.220(\sigma_n + 0.040)_{0.705}$</td>
<td></td>
</tr>
<tr>
<td>Very good quality rock mass—tightly interlocking undisturbed rock with unweathered joints spaced at $3 \text{ m} \times 3 \text{ m}$ (CSIR rating 85; NGI rating 100)</td>
<td>$\sigma_{ls} = \sigma_{sn} + \sqrt[3]{3.5\sigma_{sn} + 0.1}$</td>
<td>$\sigma_{ls} = \sigma_{sn} + \sqrt[3]{5\sigma_{sn} + 0.1}$ &amp;</td>
<td>$\sigma_{ls} = \sigma_{sn} + \sqrt[3]{7.5\sigma_{sn} + 0.1}$ &amp;</td>
<td>$\sigma_{ls} = \sigma_{sn} + \sqrt[3]{8.5\sigma_{sn} + 0.1}$ &amp;</td>
<td>$\sigma_{ls} = \sigma_{sn} + \sqrt[3]{12.5\sigma_{sn} + 0.1}$ &amp;</td>
</tr>
<tr>
<td>$\tau_s = 0.651(\sigma_n + 0.028)_{0.679}$</td>
<td>$\tau_s = 0.739(\sigma_n + 0.020)_{0.692}$</td>
<td>$\tau_s = 0.848(\sigma_n + 0.013)_{0.702}$</td>
<td>$\tau_s = 0.883(\sigma_n + 0.012)_{0.705}$</td>
<td>$\tau_s = 0.998(\sigma_n + 0.008)_{0.712}$</td>
<td></td>
</tr>
<tr>
<td>Good quality rock mass—fresh to slightly weathered rock, slightly disturbed with joints spaced at $1 \text{ m} \times 3 \text{ m}$ (CSIR rating 65; NGI rating 10)</td>
<td>$\sigma_{ls} = \sigma_{sn} + \sqrt[3]{0.7\sigma_{sn} + 0.0004}$</td>
<td>$\sigma_{ls} = \sigma_{sn} + \sqrt[3]{1.0\sigma_{sn} + 0.0004}$</td>
<td>$\sigma_{ls} = \sigma_{sn} + \sqrt[3]{1.5\sigma_{sn} + 0.0004}$</td>
<td>$\sigma_{ls} = \sigma_{sn} + \sqrt[3]{1.7\sigma_{sn} + 0.0004}$</td>
<td>$\sigma_{ls} = \sigma_{sn} + \sqrt[3]{2.5\sigma_{sn} + 0.0004}$</td>
</tr>
<tr>
<td>$\tau_s = 0.651(\sigma_n + 0.028)_{0.669}$</td>
<td>$\tau_s = 0.427(\sigma_n + 0.004)_{0.683}$</td>
<td>$\tau_s = 0.501(\sigma_n + 0.003)_{0.695}$</td>
<td>$\tau_s = 0.525(\sigma_n + 0.002)_{0.698}$</td>
<td>$\tau_s = 0.603(\sigma_n + 0.002)_{0.707}$</td>
<td></td>
</tr>
<tr>
<td>Fair quality rock mass—several sets of moderately weathered joints spaced at $0.3 \text{ m} \times 1 \text{ m}$ (CSIR rating 44; NGI rating 10)</td>
<td>$\sigma_{ls} = \sigma_{sn} + \sqrt[3]{0.14\sigma_{sn} + 0.00001}$</td>
<td>$\sigma_{ls} = \sigma_{sn} + \sqrt[3]{0.20\sigma_{sn} + 0.00001}$</td>
<td>$\sigma_{ls} = \sigma_{sn} + \sqrt[3]{0.30\sigma_{sn} + 0.00001}$</td>
<td>$\sigma_{ls} = \sigma_{sn} + \sqrt[3]{0.34\sigma_{sn} + 0.00001}$</td>
<td>$\sigma_{ls} = \sigma_{sn} + \sqrt[3]{0.50\sigma_{sn} + 0.00001}$</td>
</tr>
<tr>
<td>$\tau_s = 0.198(\sigma_n + 0.00007)_{0.682}$</td>
<td>$\tau_s = 0.234(\sigma_n + 0.00005)_{0.675}$</td>
<td>$\tau_s = 0.280(\sigma_n + 0.00003)_{0.688}$</td>
<td>$\tau_s = 0.295(\sigma_n + 0.00003)_{0.691}$</td>
<td>$\tau_s = 0.346(\sigma_n + 0.00002)_{0.700}$</td>
<td></td>
</tr>
<tr>
<td>Poor quality rock mass—numerous weathered joints spaced at $30 \text{ mm} \times 500 \text{ mm}$ with some gouge filling/clean waste rock (CSIR rating 23; NGI rating 0.1)</td>
<td>$\sigma_{ls} = \sigma_{sn} + \sqrt[3]{0.04\sigma_{sn} + 0.00001}$</td>
<td>$\sigma_{ls} = \sigma_{sn} + \sqrt[3]{0.05\sigma_{sn} + 0.00001}$</td>
<td>$\sigma_{ls} = \sigma_{sn} + \sqrt[3]{0.08\sigma_{sn} + 0.00001}$</td>
<td>$\sigma_{ls} = \sigma_{sn} + \sqrt[3]{0.09\sigma_{sn} + 0.00001}$</td>
<td>$\sigma_{ls} = \sigma_{sn} + \sqrt[3]{0.13\sigma_{sn} + 0.00001}$</td>
</tr>
<tr>
<td>$\tau_s = 0.115(\sigma_n + 0.00002)_{0.646}$</td>
<td>$\tau_s = 0.129(\sigma_n + 0.00002)_{0.655}$</td>
<td>$\tau_s = 0.162(\sigma_n + 0.00001)_{0.672}$</td>
<td>$\tau_s = 0.172(\sigma_n + 0.00001)_{0.676}$</td>
<td>$\tau_s = 0.203(\sigma_n + 0.00001)_{0.686}$</td>
<td></td>
</tr>
<tr>
<td>Very poor quality rock mass—numerous heavily weathered joints spaced less than $50 \text{ mm}$ with gouge filling/waste rock with fines (CSIR rating 3; NGI rating 0.01)</td>
<td>$\sigma_{ls} = \sigma_{sn} + \sqrt[3]{0.007\sigma_{sn} + 0}$</td>
<td>$\sigma_{ls} = \sigma_{sn} + \sqrt[3]{0.010\sigma_{sn} + 0}$</td>
<td>$\sigma_{ls} = \sigma_{sn} + \sqrt[3]{0.015\sigma_{sn} + 0}$</td>
<td>$\sigma_{ls} = \sigma_{sn} + \sqrt[3]{0.017\sigma_{sn} + 0}$</td>
<td>$\sigma_{ls} = \sigma_{sn} + \sqrt[3]{0.025\sigma_{sn} + 0}$</td>
</tr>
<tr>
<td>$\tau_s = 0.042(\sigma_n)_{0.634}$</td>
<td>$\tau_s = 0.050(\sigma_n)_{0.599}$</td>
<td>$\tau_s = 0.061(\sigma_n)_{0.546}$</td>
<td>$\tau_s = 0.065(\sigma_n)_{0.548}$</td>
<td>$\tau_s = 0.078(\sigma_n)_{0.556}$</td>
<td></td>
</tr>
</tbody>
</table>
Experience gained from the use of the Hoek-Brown criterion suggested the original values for $m$ and $s$ underestimate rockmass strength in practical applications. This was attributed to the “disturbed” quality of the foundational Panguna Andesite test samples, where the interlocking quality of the rockmass had been destroyed before testing (Brown and Hoek, 1988). Back analysis of rockmass strengths from several cases suggested different $m$ and $s$ values more appropriate for “undisturbed” rockmasses where mechanical excavation or good quality blasting maintain rockmass interlock. The relationships between $m$, $s$, and RMR for disturbed rockmasses presented by Brown and Hoek (1988) and Hoek and Brown (1988) are as follows:

\[
\frac{m}{m_i} = \exp\left(\frac{RMR - 100}{14}\right)
\]

(2.6)

\[
s = \exp\left(\frac{RMR - 100}{6}\right)
\]

(2.7)

Moreover, for undisturbed or interlocking rockmasses:

\[
\frac{m}{m_i} = \exp\left(\frac{RMR - 100}{28}\right)
\]

(2.8)

\[
s = \exp\left(\frac{RMR - 100}{9}\right)
\]

(2.9)

where $m_i$ is the value of $m$ for the intact rock. Undisturbed and disturbed rockmass assessments for the cases originally examined by Hoek and Brown (1980) are shown in Figure 2-6.
<table>
<thead>
<tr>
<th></th>
<th>CARBONATE ROCKS WITH WELL DEVELOPED CRYSTAL CLEARANCE</th>
<th>LITHIFIED ARGILLACEOUS ROCKS WITH MODERATELY ALLOTTED CLAY, LIQUID LIMIT AND PLASTICITY</th>
<th>ARENAECIOUS ROCKS WITH POORLY DEVELOPED CRYSTAL CLEARANCE</th>
<th>FINE GRAINED POLYMORPHIC CRYSTAL-LINE ROCKS</th>
<th>COARSE GRAINED POLYMORPHIC CRYSTAL-LINE ROCKS - AMPHIBOLIC, GABBRO-GNEISE, GRANITE, GRANODIORITE, ETC.</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>INTACT ROCK SAMPLES</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Laboratory size specimen free</td>
<td>m: 7.00</td>
<td>m: 10.00</td>
<td>m: 15.00</td>
<td>m: 17.00</td>
<td>m: 25.00</td>
</tr>
<tr>
<td>from discontinuities</td>
<td>s: 1.00</td>
<td>s: 1.00</td>
<td>s: 1.00</td>
<td>s: 1.00</td>
<td>s: 1.00</td>
</tr>
<tr>
<td>CSIR rating: RMR = 100</td>
<td>m: 7.00</td>
<td>m: 10.00</td>
<td>m: 15.00</td>
<td>m: 17.00</td>
<td>m: 25.00</td>
</tr>
<tr>
<td>NGI rating: Q = 500</td>
<td>s: 1.00</td>
<td>s: 1.00</td>
<td>s: 1.00</td>
<td>s: 1.00</td>
<td>s: 1.00</td>
</tr>
<tr>
<td><strong>VERY GOOD QUALITY ROCK MASS</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tightly interlocking undisturbed rock</td>
<td>m: 2.40</td>
<td>m: 3.43</td>
<td>m: 5.14</td>
<td>m: 5.82</td>
<td>m: 8.56</td>
</tr>
<tr>
<td>with unweathered joints at 1 to 3m.</td>
<td>s: 0.082</td>
<td>s: 0.082</td>
<td>s: 0.062</td>
<td>s: 0.082</td>
<td>s: 0.082</td>
</tr>
<tr>
<td>CSIR rating: RMR = 85</td>
<td>m: 4.10</td>
<td>m: 5.85</td>
<td>m: 8.78</td>
<td>m: 9.95</td>
<td>m: 14.63</td>
</tr>
<tr>
<td>NGI rating: Q = 100</td>
<td>s: 0.169</td>
<td>s: 0.189</td>
<td>s: 0.189</td>
<td>s: 0.189</td>
<td>s: 0.189</td>
</tr>
<tr>
<td><strong>GOOD QUALITY ROCK MASS</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fresh to slightly weathered rock, slightly</td>
<td>m: 0.575</td>
<td>m: 0.821</td>
<td>m: 1.231</td>
<td>m: 1.395</td>
<td>m: 2.052</td>
</tr>
<tr>
<td>disturbed with joints at 1 to 3m.</td>
<td>s: 0.00293</td>
<td>s: 0.00293</td>
<td>s: 0.00293</td>
<td>s: 0.00293</td>
<td>s: 0.00293</td>
</tr>
<tr>
<td>CSIR rating: RMR = 65</td>
<td>m: 2.066</td>
<td>m: 2.865</td>
<td>m: 4.298</td>
<td>m: 4.871</td>
<td>m: 7.163</td>
</tr>
<tr>
<td>NGI rating: Q = 10</td>
<td>s: 0.0205</td>
<td>s: 0.0205</td>
<td>s: 0.0205</td>
<td>s: 0.0205</td>
<td>s: 0.0205</td>
</tr>
<tr>
<td><strong>FAIR QUALITY ROCK MASS</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Several sets of moderately weathered</td>
<td>m: 0.128</td>
<td>m: 0.183</td>
<td>m: 0.275</td>
<td>m: 0.311</td>
<td>m: 0.458</td>
</tr>
<tr>
<td>joints spaced at 0.3 to 1 m.</td>
<td>s: 0.00009</td>
<td>s: 0.00009</td>
<td>s: 0.00009</td>
<td>s: 0.00009</td>
<td>s: 0.00009</td>
</tr>
<tr>
<td>CSIR rating: RMR = 44</td>
<td>m: 0.947</td>
<td>m: 1.353</td>
<td>m: 2.030</td>
<td>m: 2.501</td>
<td>m: 3.383</td>
</tr>
<tr>
<td>NGI rating: Q = 1</td>
<td>s: 0.00198</td>
<td>s: 0.00198</td>
<td>s: 0.00198</td>
<td>s: 0.00198</td>
<td>s: 0.00198</td>
</tr>
<tr>
<td><strong>POOR QUALITY ROCK MASS</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Numerous weathered joints at 30-500mm.</td>
<td>m: 0.029</td>
<td>m: 0.041</td>
<td>m: 0.061</td>
<td>m: 0.069</td>
<td>m: 0.102</td>
</tr>
<tr>
<td>some gogae. Clean compacted waste rock</td>
<td>s: 0.000003</td>
<td>s: 0.000003</td>
<td>s: 0.000003</td>
<td>s: 0.000003</td>
<td>s: 0.000003</td>
</tr>
<tr>
<td>CSIR rating: RMR = 23</td>
<td>m: 0.447</td>
<td>m: 0.639</td>
<td>m: 0.599</td>
<td>m: 1.087</td>
<td>m: 1.598</td>
</tr>
<tr>
<td>NGI rating: Q = 0.1</td>
<td>s: 0.00019</td>
<td>s: 0.00019</td>
<td>s: 0.00019</td>
<td>s: 0.00019</td>
<td>s: 0.00019</td>
</tr>
<tr>
<td><strong>VERY POOR QUALITY ROCK MASS</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Numerous heavily weathered joints spaced &lt;50mm with gouge. Waste rock with fines.</td>
<td>m: 0.007</td>
<td>m: 0.010</td>
<td>m: 0.015</td>
<td>m: 0.017</td>
<td>m: 0.025</td>
</tr>
<tr>
<td>CSIR rating: RMR = 3</td>
<td>m: 0.219</td>
<td>m: 0.313</td>
<td>m: 0.469</td>
<td>m: 0.532</td>
<td>m: 0.782</td>
</tr>
<tr>
<td>NGI rating: Q = 0.01</td>
<td>s: 0.00002</td>
<td>s: 0.00002</td>
<td>s: 0.00002</td>
<td>s: 0.00002</td>
<td>s: 0.00002</td>
</tr>
</tbody>
</table>

Figure 2-6: Approximate relationship between rockmass quality and Hoek-Brown material constants (Hoek and Brown, 1988)
A potential problem in using the Hoek-Brown criterion for rockmass strength estimation with the RMR or Q classification systems as input parameters is the issue of double counting in situ stress and groundwater on both sides of the analysis. To avoid this issue, Hoek and Brown (1988) recommended an RMR (Bieniawski, 1976) groundwater rating of 10 (15 when using the 1989 version of RMR (Bieniawski, 1989)), representing completely dry conditions, since groundwater pressure is otherwise accounted for in stresses acting on the rockmass. Their recommendation for the joint orientation rating is a value of zero (very favourable joint orientation) since the Hoek-Brown criterion is designed for use in cases where the rockmass exhibits isotropic behaviour. This value is termed RMR’ (Hoek and Brown, 1988). Similar recommendations were made for the Q system (Barton et al., 1974), where the joint water reduction factor (Jw) and stress reduction factor (SRF) should be set to unity and therefore do not influence the other Q parameters, which is known as Q’ (Hoek and Brown, 1988). These stress, groundwater, and joint orientation parameters were instead intended to be separately incorporated into numerical models.

Field applications of the original Hoek-Brown criterion show adequate strength prediction of good to reasonable quality rocks at a range of confining stresses and poor quality rocks at moderate compressive confinement. Applications of the criterion for poor quality rocks at low (compressive) to negative (tensile) confining stresses, such as failure in immediate proximity to an underground excavation boundary, show an overestimation of rockmass strength (Shah, 1992). To address this issue, Shah (1992) developed a Modified Hoek-Brown criterion that has no s parameters, following the observation that poor quality rockmasses have zero cohesion. In addition, Shah (1992) implemented the modification of the square root to a variable exponent parameter, a, which was first proposed by Pan and Hudson (1988) to improve curve fits of the original Hoek-Brown criterion for weaker rocks. The Modified Hoek-Brown criterion (Shah, 1992) is shown in Equation 2.10. Removal of the s parameter rather than forcing a fit of s = 0 in the original criterion met the requirement for rockmasses to have zero tensile strength while maintaining an accurate estimate of rockmass strength over a range of confinements. Furthermore, the
Modified Hoek-Brown criterion maintained accuracy from the original criterion in moderate to high confining stresses with the flexibility of curvature provided by $a$.

$$
\sigma'_1 = \sigma'_3 + \sigma_c \left( m_b \frac{\sigma'_3}{\sigma_c} \right)^a
$$

(2.10)

Strength characteristics that are major controls of rockmass strength include block shape and size and the surface condition of the intersecting discontinuities (Hoek et al., 1992). In general, large, angular blocks with very rough and fresh surfaces correlate to very good rockmass strength, while very small, broken blocks with highly weathered and infilled surfaces correlate to very poor rockmass strength. These considerations were first organized by Hoek et al. (1992) into a classification system to estimate values of the $m_b/m_i$ and $a$ parameters for Shah’s (1992) Modified Hoek-Brown criterion, as shown in Figure 2-7.

The most general form of the Hoek-Brown criterion that accommodates both good and poor quality rockmasses was proposed by Hoek (1994) and Hoek et al. (1995) (in essentially the same discussion), where it maintains a combination of the $m_b$, $s$, and $a$ parameters:

$$
\sigma'_1 = \sigma'_3 + \sigma_c \left( m_b \frac{\sigma'_3}{\sigma_c} + s \right)^a
$$

(2.11)

where $\sigma'_1$ and $\sigma'_3$ are axial and confining effective principal stresses, $\sigma_c$ is the uniaxial compressive strength of the intact rock material, and the constant $m_b$ applies to the rockmass. It has since become known as the Generalized Hoek-Brown criterion (Hoek et al., 2002) and it remains the current and most popular form. Established practices for laboratory testing of intact rock (e.g. ISRM, 1978a; ISRM, 1978b; ISRM, 1983) make for a straightforward application of the Hoek-Brown criterion. For laboratory testing of intact rock, $s = 1$, $a = 0.5$, and $m_b$ is replaced with $m_i$ and is defined by the ratio of uniaxial compressive strength to tensile strength.

Trends of $m_i$ by lithology determined by examination of laboratory test results indicate the lowest values are attributed to fine-grained sedimentary rocks, and the highest values describe coarse-grained
igneous and metamorphic rocks with interlocked crystal structure and silicate mineralogy (Hoek et al., 1992). Example values of $m_i$ for intact rock by lithology are shown in Table 2-3 (Hoek and Brown, 1997).

Figure 2-7: Estimation of $m_i/m_b$ and $a$ based on rock structure and surface condition (Hoek et al., 1992)
Table 2-3: Values of the Hoek-Brown constant $m$ for intact rock. Note that values in parenthesis are estimates (Hoek and Brown, 1997)

<table>
<thead>
<tr>
<th>Rock type</th>
<th>Class</th>
<th>Group</th>
<th>Course</th>
<th>Medium</th>
<th>Fine</th>
<th>Very fine</th>
</tr>
</thead>
<tbody>
<tr>
<td>SEDIMENTARY</td>
<td>Clastic</td>
<td>Conglomerate (22)</td>
<td>Sandstone 19</td>
<td>Silstone 9</td>
<td>Claystone 4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Non-Clastic</td>
<td>Organic</td>
<td>7</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Carbonate</td>
<td>Breccia (20)</td>
<td>Sparrtic (8-21)</td>
<td>Micritic Limestone 8</td>
<td>Anhydrite 13</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Chemical</td>
<td>Gypsum (16)</td>
<td>Limestone (10)</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>METAMORPHIC</td>
<td>Non-foliated</td>
<td>Marble 9</td>
<td>Hornfels (19)</td>
<td>Quartzite 24</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Slightly foliated</td>
<td>Migmatite (30)</td>
<td>Amphibolite 25-31</td>
<td>Mylonites (6)</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Foliated*</td>
<td>Gneiss 33</td>
<td>Schists 4-8</td>
<td>Phyllites (10)</td>
<td>Slate 9</td>
<td></td>
</tr>
<tr>
<td>IGNEOUS</td>
<td>Light</td>
<td>Granite 33</td>
<td>Rhyolite (16)</td>
<td>Obsidian (19)</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Dark</td>
<td>Granodiorite (30)</td>
<td>Dacite (17)</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Diorite (28)</td>
<td>Andesite 19</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Gabbro 27</td>
<td>Basalt (17)</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Extrusive pyroclastic type</td>
<td>Agglomerate (20)</td>
<td>Breccia (18)</td>
<td>Tuff (15)</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>

*These values are for intact rock specimens tested normal to bedding or foliation. The value of $m_i$ will be significantly different if failure occurs along a weakness plane.

While the original Hoek-Brown criterion for rockmass strength assessment primarily used the RMR classification system to estimate $m_b$, $s$, and $a$, the effectiveness of RMR was limited to moderate to good quality rockmasses with RMR values greater than around 25. This constraint led to the creation of the Geological Strength Index (GSI) rockmass characterization system to be used with the Hoek-Brown criterion which applies to a wider variety of rockmasses than RMR considers (Hoek, 1994; Hoek et al., 1995). Unlike RMR and Q, GSI was designed to use direct descriptions of geological field observations. The goal was to create an intuitive system for qualified and experienced geologists and geological engineers to assess rockmass strength in the field and then apply the data to the Hoek-Brown strength criterion in numerical analyses.
The first range of GSI values for rockmasses was from 10 for extremely poor quality to 85 for very good quality, while a single value of 100 represented intact rock (Hoek, 1994). The relationships between the Hoek-Brown criterion parameters \( (m_b, m_i, s, a) \) and GSI are shown in Equations 2.12 to 2.18 (Hoek et al., 1995). For GSI > 25 (undisturbed rockmasses):

\[
\frac{m_b}{m_i} = \exp\left(\frac{GSI - 100}{28}\right) \quad (2.12)
\]

\[
s = \exp\left(\frac{GSI - 100}{9}\right) \quad (2.13)
\]

\[a = 0.5 \quad (2.14)\]

For GSI < 25 (disturbed rockmasses):

\[s = 0 \quad (2.15)\]

\[a = 0.65 - \frac{GSI}{200} \quad (2.16)\]

In cases where GSI values from direct observation are absent or as a secondary check, RMR\(_{89}\) and RMR\(_{76}\) can be converted into GSI using the following relationships (Hoek et al., 1995):

\[GSI = RMR'_{89} - 5 \quad \text{(where } RMR'_{89} > 23) \quad (2.17)\]

\[GSI = RMR'_{76} \quad \text{(where } RMR'_{76} > 18) \quad (2.18)\]

For RMR\(_{89}^' < 23 \) (and for RMR\(_{76}^' < 18 \)), RMR’ cannot be used to estimate GSI; the Q’ value (Barton et al., 1974) should be used instead:

\[GSI = 9 \log_e Q' + 44 \quad (2.19)\]

Estimates of GSI and related parameters in the Generalized Hoek-Brown criterion were organized in a now familiar fashion that characterizes rockmass structure (block size and interlocking) and discontinuity surface condition (Hoek et al., 1995), following a similar chart by Hoek et al. (1992) (see Figure 2-8). The contoured field estimation chart of GSI based on geological descriptions was presented shortly thereafter (Hoek and Brown, 1997) (see Figure 2-9). Although the GSI system was not formally defined until 1994, the roots of a rockmass characterization system based on observable characteristics were first established in 1992.
Figure 2-8: Estimation of constants $m_b / m_i$, $s$, $a$, Young’s modulus ($E$), Poisson’s Ratio ($\nu$), and GSI for the Generalized Hoek-Brown strength criterion based on rockmass structure and discontinuity surface conditions. Note that the values given in this table are for an undisturbed rockmass (Hoek et al., 1995)
Figure 2-9: Contoured field estimation chart of Geological Strength Index (GSI) based on geological descriptions (Hoek and Brown, 1997)
While the Generalized Hoek-Brown criterion presented by Hoek et al. (1995) and shown here in Equation 2.11 remains the current format, a significant update to the input parameters by Hoek et al. (2002) replaced the arbitrary boundary of GSI = 25 between undisturbed and disturbed rockmasses with a general Damage Factor (D). These current relationships between the Hoek-Brown criterion parameters \((m_b, m_i, s, a, \text{ and } D)\) and GSI are shown in Equations 2.20 to 2.22.

\[
m_b = m_i \exp \left( \frac{GSI - 100}{28 - 14D} \right) \tag{2.20}
\]

\[
s = \exp \left( \frac{GSI - 100}{9 - 3D} \right) \tag{2.21}
\]

\[
a = \frac{1}{2} + \frac{1}{6} \left( e^{-GSI/15} - e^{-20/3} \right) \tag{2.22}
\]

where D describes the amount of disturbance in the rockmass due to stress damage and stress relaxation (Hoek et al., 2002). D varies from 0 for undisturbed in situ rockmasses to 1 for very disturbed rockmasses; specific guidelines for the use of D are discussed by Hoek et al. (2002). It is important to note that while \(a\) is variable, for moderate to high quality rockmasses (and GSI ratings), \(a\) is approximately 0.5, while for poor quality rockmasses (and low GSI ratings), \(a\) increases to 0.6.

The effectiveness of the original GSI system was limited to reasonably competent rockmasses. The very poor quality rockmasses that commonly occur in Greece, such as thinly foliated, folded, and sheared weak rocks of non-blocky structure, did not fall within the criteria for the original GSI. This limitation resulted in an expansion of the system to accommodate these rocks, where the category for laminated/sheared structure was added, extending the lower bound from 10 to 5 (Hoek et al., 1998). On the high end of the scale, a category for massive rockmass structure was added to extend the upper bound from 85 to 100 (Hoek and Marinos, 2000). These updates have resulted in a qualitative GSI chart that remains in common practice (Figure 2-10). Further work conducted with the boon of numerous tunnelling projects in Greece during the early 2000s detailed thousands of designed and observed rockmass characterizations with GSI, excavation designs, rockmass behaviour during excavation, and effective
ground support measures. The Tunnel Behaviour Chart (TBC) by Marinos (2012) synthesizes this data into a useful tool for predicting rockmass behaviour for a variety of rockmass structures (defined using GSI), intact rock strength, and in situ stresses (see Figure 2-11 and Figure 2-12). The TBC is limited to experience from the examined Greek tunnelling cases including maximum overburden depths of 500 m and it does not consider spalling or other brittle failure of massive rockmasses with minimal structure.

Figure 2-10: Most recent qualitative GSI chart with added massive and laminated/sheared categories (Hoek and Marinos, 2000)
### TUNNEL BEHAVIOUR TYPES

<table>
<thead>
<tr>
<th>Code</th>
<th>Type</th>
<th>Description</th>
<th>Diagram</th>
</tr>
</thead>
<tbody>
<tr>
<td>St</td>
<td>Stable ground</td>
<td>Stable tunnel section with low gravity hazards. Rock mass is compact with limited and isolated discontinuities</td>
<td><img src="image" alt="St" /></td>
</tr>
<tr>
<td>Br</td>
<td>Brittle failure</td>
<td>Brittle failure or rock bursting at great depths</td>
<td><img src="image" alt="Br" /></td>
</tr>
<tr>
<td>Wg</td>
<td>Wedge failure</td>
<td>Wedge sliding or gravity driven failures. Insignificant strains. Rock mass is blocky to very blocky. Blocks can fall or slide. The stability is controlled by the geometrical and mechanical characteristics of the discontinuities. The ratio of rock mass strength to the in situ stress ($\sigma_{\text{in situ}}$) is high ($&gt;0.6$) and thin at very small stresses ($&lt;0.1$).</td>
<td><img src="image" alt="Wg" /></td>
</tr>
<tr>
<td>Ch</td>
<td>Charnock type failure</td>
<td>Rock mass is highly fractured, retaining most of the intact rock structure or at least that of the surrounding rock mass. Rock mass does not have good interlocking (open structure) and in combination with low confinement (tensile stress), rock mass can generate high distributed stress in the rock mass. The difference with Ch type lies in the block size, which is very small here, the self-supporting capacity, which is very limited here and the failure extension, where it is unrestricted due to the lack of better rock mass quality in the surrounding zone.</td>
<td><img src="image" alt="Ch" /></td>
</tr>
<tr>
<td>Rv</td>
<td>Raveling ground</td>
<td>The rock mass is extensively fissured or failed with practically zero cohesion and negligible strength. The rock mass can generate high distributed stress in the rock mass and may be affected by the self-supporting capacity, which is very limited here and the failure extension, where it is unrestricted due to the lack of better rock mass quality in the surrounding zone.</td>
<td><img src="image" alt="Rv" /></td>
</tr>
<tr>
<td>Fl</td>
<td>Flowing ground</td>
<td>The rock mass is extensively fissured or failed with practically zero cohesion and negligible strength. Rock strain is low, and the failure extension, where it is unrestricted due to the lack of better rock mass quality in the surrounding zone.</td>
<td><img src="image" alt="Fl" /></td>
</tr>
<tr>
<td>Sh</td>
<td>Shear failure</td>
<td>Minor to medium strains, with the development of shear failure close to the perimeter around the tunnel. Rock mass is characterized by low strength intact rocks ($\sigma_{\text{intact}} &lt; 150$ MPa) while the rock mass structure reduces the overall rock mass strength.</td>
<td><img src="image" alt="Sh" /></td>
</tr>
<tr>
<td>Sq</td>
<td>Squeezing ground</td>
<td>Large strain, due to overpressuring with the development of shear failures in an excavated zone around the tunnel. Rock mass consists of low strength intact rocks while the rock mass structure reduces the overall rock mass strength. The ratio of rock mass strength to the in situ stress ($\sigma_{\text{in situ}}$) is very low ($&lt;0.3$) and strains are measured or expected to be $&gt;2.5$ %, and they can be also take place at the face.</td>
<td><img src="image" alt="Sq" /></td>
</tr>
<tr>
<td>Sw</td>
<td>Swelling ground</td>
<td>Rock mass contains a significant amount of swelling minerals (montmorillonite, smectite, anhydrite) which swell and deform in the presence of groundwater. Swelling often occurs in the tunnel floor when the support ring is not fully closed.</td>
<td><img src="image" alt="Sw" /></td>
</tr>
<tr>
<td>San</td>
<td>Anisotropic strains</td>
<td>The rock mass is stratified or schistose or consists of specific weak zones and develops increased strain characteristics along a direction defined by the schistosity.</td>
<td><img src="image" alt="San" /></td>
</tr>
</tbody>
</table>

Figure 2-11: Brief description and schematic diagrams of tunnel behaviour types used in the Tunnel Behaviour Chart that is shown in Figure 2-12 (Marinos, 2012)
**Figure 2-12: Tunnel Behaviour Chart (TBC): An assessment of rockmass behaviour in tunnelling from experience in Greece (Marinos, 2012)**

<table>
<thead>
<tr>
<th>ROCK MASS STRUCTURE</th>
<th>OVERBURDEN (H)</th>
<th>ROCK masses for up to several hundreds metres**</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Small overburden</td>
<td>Large overburden</td>
</tr>
<tr>
<td></td>
<td>INTACT ROCK STRENGTH ($\sigma_r$)</td>
<td>INDICATIVE LIMIT $\sigma_r$</td>
</tr>
<tr>
<td></td>
<td>Low $\sigma_r$</td>
<td>High $\sigma_r$</td>
</tr>
<tr>
<td>INTACT OR MASSIVE</td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>Intact rock specimen or massive in situ rock with few widely spaced discontinuities</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BLOCKY</td>
<td>5</td>
<td>6</td>
</tr>
<tr>
<td>Well interlocked disturbed rock mass consisting of blocks formed by three orthogonal intersecting discontinuity sets</td>
<td></td>
<td></td>
</tr>
<tr>
<td>VERY BLOCKY</td>
<td>9</td>
<td>10</td>
</tr>
<tr>
<td>Interlocked partially disturbed rock mass with multi-oriented angular blocks formed by four or more discontinuity sets</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BLOCKY/DISTURBED/SAMMY</td>
<td>13</td>
<td>14</td>
</tr>
<tr>
<td>Folded with angular blocks formed by many intersecting discontinuity sets. Persistence of folding plane or schistosity. It is understood that the rock mass is disturbed and anisotropy can be developed</td>
<td></td>
<td></td>
</tr>
<tr>
<td>DISINTEGRATED</td>
<td>17</td>
<td>18</td>
</tr>
<tr>
<td>Poorly interlocked, heavily broken rock mass with mixture of angular and rounded rock pieces</td>
<td></td>
<td></td>
</tr>
<tr>
<td>LAMINATED/FOLIATED/SHEARED</td>
<td>21</td>
<td>22</td>
</tr>
<tr>
<td>Laminated or foliated and technically a nearly weak rock mass, foliation prevails over any other discontinuity set, resulting in complete lack of blockiness. (This grading scale is not compared with the others drawing scales)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes:
- The data used in the TBC were obtained from tunnels excavated by the conventional method with top heading and bench in a non-urban environment with the overburden cover up to several hundred metres (generally not exceeding 500m) with a tunnel diameter=12m.
- The chart does not refer to very high overburden (e.g. many hundreds of m or >1000m), where the scale and the mechanism of failure may differ.
- The limit-ranges of the uniaxial compressive strength ($\sigma_r$) of the intact rock and the overburden thickness (H) are indicative. This is done to avoid standardisation by an inexperienced user. The purpose of this diagram is to predict the failure mechanisms of several common rock mass types.
- The surface condition of discontinuities, the second component to the GSI system, mainly affect the intensity of the failure phenomenon.
- High clay presence along the discontinuities or zones in the rock mass may shift the gravity driven behaviour types towards the vertical axis of the chart (e.g. from Wg(9) to Ch(13)).
- Groundwater presence mainly affects the factor of safety and not the behaviour type. Though, in some cases, such as "Blocky-Disturbed" & "Disintegrated" rock mass, the groundwater presence may "shift" a Charnoy (Ch) or Flowing (Rv) behaviour type to Flowing ground (F).
- Cases number 4, 8 and 12 may develop brittle failures (Br) when overburden increases considerably (e.g. >800 m) depending on the intact rock strength.
- The illustrations of the tunnel are sketchy; this shape corresponds to the usual top heading.
The application of the Hoek-Brown criterion to hard, brittle rocks in the GSI range > 65 has been found to not adequately account for brittle crack initiation and damage, crack propagation and minimal frictional strength in low confinement conditions (Martin et al., 1999; Diederichs, 2007). While the failure of massive rockmasses is controlled by intact rock strength, the in situ intact rock strength is a fraction of the laboratory UCS due to the promotion of shear failure and suppression of extension cracks in these tests. The geometrically unrestricted development of extension cracks in situ leads to a lower bound strength of spalling failure near excavations at deviatoric stresses between 30-50% of the upper bound laboratory UCS (Diederichs, 2007). The slope of the lower bound failure envelope for damage initiation in principal stress space is low compared to typical Hoek-Brown shear failure of less competent rockmasses. The transition between the lower bound damage initiation (field) and upper bound crack interaction (lab) strengths is controlled by the spalling limit (ratio of $\sigma_3/\sigma_1$) that divides low confinement, uncontrolled extension crack propagation from high confinement shear failure through the accumulation and coalescence of cracks (Diederichs, 2003). Hoek (1968) advised this spalling limit occurs between approximately 0.1 and 0.05.

Diederichs (2007) proposed a methodology to apply the Generalized Hoek-Brown criterion to hard, brittle rockmasses with sparse jointing (Massive) that captures the lower bound systematic damage initiation envelope as “peak” strength and the spalling limit as the “residual” strength envelope. This methodology is also known as the damage initiation – spalling limit (DISL) approach for rock strength (see Figure 2-13). The key modifications to the Hoek-Brown criterion for brittle failure do not impose the conventional limits on the $m$, $s$, and $a$ parameters. For instance, the curvature of the envelope (defined by $a$) is the primary control on an appropriate DISL envelope, where the damage initiation “peak” envelope can have values for $a$ as low as 0.2. The spalling limit “residual” envelope reaches $a = 0.75$ for the example of Lac du Bonnet granite in Figure 2-13. Indeed, the modifications to the Hoek-Brown criterion since its inception have been invaluable for promoting the flexibility of the system for a wide variety of rockmasses.
Figure 2-13: Generalized Hoek-Brown composite strength envelopes using the DISL approach for brittle failure of Lac du Bonnet granite, where the laboratory-measured tensile strength is assumed to anchor the damage initiation and systematic damage envelopes (Diederichs, 2007)

2.3.1 Quantifications of GSI

Multiple authors have developed quantified modifications to GSI, including Sonmez and Ulusay (1999), Cai et al. (2004), and Hoek et al. (2013), in response to challenges of subjectivity faced by practitioners with different levels of experience using the qualitative GSI system. These quantifications provide more objective definitions of GSI inputs using a numerical basis to improve communication between practitioners. Sonmez and Ulusay (1999) proposed the structure rating (SR) based on volumetric joint count (joints/m$^3$) and surface condition rating (SCR), estimated from discontinuity characteristics such as
roughness, weathering and infilling. This quantification was tested for validity using case histories of slope instabilities in Turkey. Cai et al. (2004) suggested a quantification of structure based on the mean discontinuity spacing (S) or by the mean block volume \( V_b \) and a quantification of surface condition similar to the joint condition factor \( J_c \) coefficient used by Palmstrøm (1996) in the RMi classification system. Where there are at least three joint sets, the mean block volume \( V_b \) can be calculated (see Figure 2-14) using the joint spacing \( S_i \) and the angles between joint sets \( \gamma_i \) (Palmstrøm, 1996):

\[
V_b = \frac{S_1 S_2 S_3}{\sin \gamma_1 \sin \gamma_2 \sin \gamma_3}
\]  

(2.23)

Compared to the variation in joint spacing, the effect of the joint intersection angle is minimal, so for practical purposes, the block volume \( V_b \) can be approximated as (Cai et al., 2004):

\[
V_b = S_1 S_2 S_3
\]

(2.24)

For non-persistent or irregular joint sets, Cai et al. (2004) suggest that direct measurement of representative blocks in the field is sufficient. The quantified GSI chart by Cai et al. (2004) is shown in Figure 2-15.

![Figure 2-14: Block volume calculation components in a block with three joint sets (Cai et al., 2004)]
Figure 2-15: Quantified GSI chart (modified after Cai et al., 2004)
The GSI chart was revisited by the original authors to quantify the inputs and improve the uniformity for more effective implementation in numerical models when coupled with the Hoek-Brown strength criterion (Hoek et al., 2013). In this updated chart, GSI values can be determined quantitatively by summing the two linear scales that represent the discontinuity surface conditions (scale A) and the interlocking of rock blocks defined by these intersecting discontinuities (scale B) (see Figure 2-16). The ratings used to quantify the A and B scales must be from systems “that are familiar to engineering geologists and geotechnical engineers operating in the field” (Hoek et al., 2013). An example quantification of scales A and B presented and tested by Hoek et al. (2013) use the “boringly reliable” Rock Quality Designation (RQD) by Deere et al. (1969) for rockmass structure, and the Joint Condition (JCond\textsubscript{89}) rating defined by Bieniawski (1989) for the discontinuity surface condition in the following relationship:

\[ GSI = 1.5JCond_{89} + RQD/2 \] (2.25)

The selection of appropriate quantities is dependent on the available field data for a given project and the experience of the involved personnel. This updated chart is designed to be flexible for user preferences in both the qualitative camp, where GSI is estimated from direct field observations of rockmasses, and the quantitative camp.

An important difference between the updated GSI chart by Hoek et al. (2013) and the version by Hoek and Marinos (2000), is the removal of the Massive and Laminated/Sheared bins of rockmass structure to be true to the fundamental assumption of the GSI system that rockmass deformation and strength are controlled by sliding and rotation of intact blocks of rock defined by intersecting discontinuities, and to account for micro-defects in the rock between laboratory testing and field scales (Hoek et al., 2013). Furthermore, it is assumed there are several sets of discontinuities and their spacing, relative to the excavation under consideration, which results in a homogeneous and isotropic rockmass (see Figure 2-17).
Figure 2-16: Updated GSI chart by Hoek et al. (2013), where the contour grid has been linearized, and the structure and surface condition scales can be quantified with user-defined axes (scales A and B)
Removal of the Massive bin excludes brittle failure processes while removal of the Laminated/Sheared bin excludes disturbed materials that encroach on the realm of soil mechanics. For consideration of disturbed materials such as flysch, users are directed to specific GSI charts by Marinos and Hoek (2001). While the maximum GSI for rockmasses in the 2013 chart is 85, a GSI of 100 is still intended for intact rock in the Hoek-Brown strength criterion.

Figure 2-17: Appropriate use and limitations of GSI depending on scale (Hoek et al., 2013)
2.4 References


Chapter 3

A New Composite GSI Approach and Applications to Intrablock Structure

3.1 Introduction

The GSI and the Hoek-Brown criterion continue to be effective methods to assess conventional rockmasses comprised of intact rock (micro-scale structure) and fractures (macro-scale structure). However, unconventional and complex rockmasses that contain meso-scale healed structure, when coupled with deeper modern excavations, present another challenge. Meso-scale healed structure, such as hydrothermal veins, veinlets, stockwork, and lithified sediment disturbance features, exists within blocks bounded by macro-scale structures such as joints, bedding, and other fractures. This meso-scale healed structure has been termed intrablock structure where macro-scale fractures are termed interblock structure. Examples of healed intrablock structure in fragmented blocks (observed in an underground drift) and drill core are shown in Figure 3-1. Intrablock structure has been traditionally considered to be a seamless component of the intact rock and has a negligible effect on rockmass behaviour for shallow excavations with simple geometries. However, healed intrablock structure has been observed in several cases of deeper excavations with higher and more complex stress paths, as well as laboratory tests, to have a significant influence on rockmass behaviour and mechanical properties. Intrablock structure must, therefore, be considered in modern rockmass characterization and geotechnical design.

Methods to estimate the strength of a complex rockmass that contains both interblock and intrablock structure are developed in this work, using Finite Element Method (FEM) numerical tools. Input properties are determined using GSI and field observations at the rockmass structure (or excavation) scale, in addition to FEM numerical tools coupled with the Hoek-Brown criterion and laboratory test results at the intact drill core sample scale. At the rockmass structure scale, a new GSI chart is developed

---

1 Parts of this chapter are in preparation for submission to an international journal with the following author list: Day, J. J., Diederichs, M. S., and Hutchinson, D. J.
to include evaluations of intrablock structure and the new Composite GSI (CGSI) approach introduces a methodology to evaluate complex rockmasses with multiple suites of structure. A numerical study is used to illustrate the improvements of CGSI in rockmass strength estimation from the conventional GSI approach. Two case examples are used to illustrate a procedure for optimizing numerical simulation of explicit rockmass structure in cases that have multiple suites of structure. The CGSI method is also used to analyze validation case studies of an adit and drift at the El Teniente copper porphyry mine in Chile. At the intact laboratory sample scale, test results of case studies in Chile are separated to generate different strength envelopes by defining whether failure has occurred through the intact matrix or along intrablock structure. The objective of considering both excavation and laboratory test scales is to determine improved GSI and Hoek-Brown properties for equivalent-continuum numerical modelling of complex rockmasses. In addition, the risks of a lack of consideration for, or an erroneous assessment of, intrablock structure are discussed.
Figure 3-1: Three examples of hydrothermal vein types of intrablock structure from Chile
3.2 Applying GSI to Multiple Sets or Suites of Structure

The conventional use of the GSI system dictates that when evaluating a typical rockmass, the overall average block size of the structure and joint condition are selected and represented by a single GSI value. A conventional and conservative approach to incorporate another set or suite of structure with significantly different characteristics would be to combine the poorest quality characteristics from each block size and joint condition ranking to give an overall GSI value for the rockmass. Using the example shown in Figure 3-2, the combination of the blocky and fair condition structure suite (blue square at coordinates (BB, JB) = (5.75, 4.8)) and the very blocky and very good condition structure suite (orange diamond at (BB, JB) = (3.6, 6.0)) would give a conventional worst case GSI value of 46 (white circle at min{BB, JB} = (3.6, 4.8)). This method has been shown by field observations to underestimate rockmass strength at the excavation scale.

A new method is proposed in this thesis that calculates a more realistic GSI value, the Composite GSI (CGSI), for a rockmass that contains multiple distinct suites of structure. The new Composite GSI (CGSI) method, as applied to the GSI chart by Hoek and Marinos (2000) and quantified by Cai et al. (2004) (see Figure 3-2), is described using BB and JB rankings. JB is a simplified ranking for discontinuity surface condition with index values ranging from 1 (very poor) to 6 (very good) and BB is a simplified bin ranking for block volume with index values ranging from 0 to 7 (x where 10^x is the block volume in cm^3). For a rockmass containing multiple and distinct suites of structure, weighted composite values for JB and BB (JB* and BB*) can be obtained. JB_1, BB_1 apply to the first system (e.g. a clean, rough set of interblock structure, JB_2, BB_2 apply to the second system (e.g. an infilled, smooth set of interblock structure), JB_3, BB_3 apply to the third system (e.g. hydrothermal vein intrablock structure), and so on as in Equations 3.1 and 3.2.
Figure 3-2: Geological Strength Index (GSI) chart, modified after the Tzamos and Sofianos (2007) version (with quantifications by Cai et al., 2004 and descriptions by Hoek and Marinos, 2000), showing an example of a complex rockmass with suites of interblock (blue square at JB1, BB1) and intrablock (orange diamond at JB2, BB2) structures; the conventional worst case GSI (white circle with purple outline) and new Composite GSI (CGSI) (green filled circle at JB*, BB*) ratings of this rockmass are also shown in their calculated (JB, BB) coordinates.
\[
BB^* = \log_{10} \left( 10^{-BB_1/3} + 10^{-BB_2/3} + 10^{-BB_3/3} + \ldots + 10^{-BB_n/3} \right)^{-3} 
\]  
\( (3.1) \)

\[
JB^* = \frac{(JB_1 / BB_1) + (JB_2 / BB_2) + (JB_3 / BB_3) + \ldots + (JB_n / BB_n)}{(1 / BB_1) + (1 / BB_2) + (1 / BB_3) + \ldots + (1 / BB_n)} 
\]  
\( (3.2) \)

where BB* and JB* are equivalent blended parameters for the composite rockmass. The Composite GSI (CGSI) then is defined by the power function:

\[
CGSI = 9(JB^*) + 1.5(BB^*)^{1.4} + 0.4(JB^*)(BB^*)^{1.4} 
\]  
\( (3.3) \)

The CGSI function was fit to GSI values on the Hoek and Marinos (2000) chart using each integer coordinate of JB and BB, as shown in Figure 3-3. This power function is an acceptable fit based on an \( R^2 \) value of 0.999. The CGSI value for the example rockmass in Figure 3-2 has been calculated as 58 using Equation 3.3. The corresponding input values for JB* and BB* were calculated using Equations 3.1 and 3.2, and the results are shown by the filled green circle at (BB*, JB*) = (3.5, 5.1) in Figure 3-2.

---

**Figure 3-3:** Surface of GSI chart based on JB and BB integer coordinates compared to fitted and contoured Composite GSI (CGSI) power function
3.3 Influence of Intrablock Structures on Rockmass Strength

Most current design practices, based on either empirical or numerical approaches, do not consider the effect of intrablock structures on rockmass strength. It is commonly assumed that the rockmass strength is affected by intact strength and interblock structures such as joints and bedding; however, field evidence in deep, high stress environments has shown that intrablock structures such as veins, veinlets, and stockwork that exist in blocks of “intact” rock also, when present, have an influence on rockmass strength.

Intrablock structures are important to consider in a variety of geological environments, including hydrothermally altered volcanic settings and nodular sedimentary limestone. Hydrothermally altered rock contains a variety of associated minerals that are either disseminated in the intact rock or concentrated in multiple generations of veins, veinlets, and stockwork, or both. These minerals have a considerable range of stiffness and strength properties and can include quartz, pyrite, copper sulphides, biotite, chlorite, gypsum, anhydrite, and clay minerals, among others (Sinclair, 2007).

Intrablock structures around calcite-rich nodules in argillaceous sedimentary limestone develop from pressure dissolution during compaction (Choquette and James, 1987) or intense bioturbation during deposition in a sheltered marine environment, which allows the bioturbated material to remain undisturbed during lithification (Johnson et al., 1992).

Intrablock structure has a significant influence on rockmass shear and tensile strength in high stress environments. A distinguishing feature of intrablock structure is that it can remain intact in good quality drill core (Figure 3-4). In a rockmass under disturbance at the excavation scale, intrablock structure can control the ultimate fragmentation block size (Figure 3-4c). Ultimately, rockmass strength depends on the thickness, persistence, orientation, and mineralization of each suite of intrablock structure.
Figure 3-4: Rock from the Oyu Tolgoi Cu-Au Porphyry deposit in Mongolia. Joints (arrows) and intrablock structure (other visible traces) in core (left) and in underground rockmass (middle); ultimate fragmentation (right) shows size controlled by intrablock structure under disturbance (courtesy M. S. Diederichs)

The Mohs’ hardness scale for minerals (Mohs, 1825) correlates hardness to the mechanical behaviour of infill mineralogies of intrablock structure. A common strengthening mineral that can appear welded to the wall rock is quartz (see Figure 3-5, a-c). Other strengthening minerals include sulphides like pyrite and chalcopyrite (Figure 3-5, d-f) and strengthening quartz can also be interlaced with sulphide minerals (Figure 3-5 e). As mineral hardness decreases to minerals such as anhydrite, gypsum (Figure 3-5 g-i), epidote (Figure 3-5 j), calcite (Figure 3-5 k), biotite, muscovite (including fine-grained sericite), and clay minerals (Figure 3-5 l-m), there is a transition between effects of strengthening to weakening of the rockmass by the intrablock structure, where local variations in thickness, persistence, and orientation control the effect on the overall rockmass. The competence of the contact between the wall rock and infill mineralogy of intrablock structure is also an important factor in the overall strength. Weaker and friable infill minerals tend to have poor adhesion to the wall rock while stronger minerals can have an excellent, fused contact (see Figure 3-5 a-f versus h-m). It is important to recognize the weakening or strengthening effects and competency of various mineralogies of intrablock structure for effective excavation design and implementation of ground support.
Figure 3-5: Drill core from Chilean porphyry and Sudbury, Canada magmatic deposits showing various infill - wall rock contact qualities of intrablock structure; (a-c) strengthening welded quartz veins; (d-f) sulphide veins (pyrite, chalcopyrite) with some quartz; (g-i) healed and broken gypsum veins; (j) epidote vein that broke during drilling; (k) weak calcite vein that broke during drilling; (l-m) weak swelling clay infilling that has been altered and expanded by water application during core logging.
3.4 Incorporating Intrablock Structure into Equivalent-Continuum Numerical Design

The application of GSI and intact rock strength parameters in the Hoek-Brown criterion are modified in this work to integrate the characterization of intrablock structure, in conjunction with the present analysis of interblock structure, into rockmass strength and subsequent numerical design. The numerical design process for these techniques applies equivalent continuum models where the structure is implicitly modelled using strength parameters provided by the GSI and Hoek-Brown criterion methods. The equivalent continuum modelling technique requires relatively little time and computational capacity with current computer hardware when compared to models with fully explicit or discrete structure and is therefore well suited to preliminary stages of geotechnical design.

At the rockmass scale, the proposed Composite GSI (CGSI) handles multiple suites of rockmass structure by including intrablock structure and calculating a weighted harmonic average to provide a comprehensive, modified CGSI value for a complex rockmass. At the intact rock scale, laboratory UCS and triaxial strength test results are analyzed by separate failure modes to produce best-fit Hoek-Brown properties and failure envelopes that capture the behaviour of intrablock structure. The failure modes of laboratory testing are characterized by failure through the intact rock matrix or failure through intrablock structure.

3.5 Accounting for Intrablock Structure using GSI

The Composite GSI (CGSI) approach provides an improved estimate of strength for rockmasses that contain multiple suites of structure. While also applicable to conventional rockmasses that contain only interblock structures, the CGSI approach is particularly designed for complex rockmasses that contain multiple suites of interblock and intrablock structures in various combinations. To improve the characterization of intrablock structure using the GSI system, modifications to the quantified and linearized GSI chart by Hoek et al. (2013) have been developed and are presented here.

This new GSI chart for jointed blocky rockmasses and healed intrablock structure is presented in Figure 3-6. A column is added to the description of discontinuity surface conditions to include
strengthening intrablock structure with very good wall rock adhesion. In addition, the description of the existing conditions was modified to include a range of intrablock structure with variable competence and strength that overlaps with very good to fair quality joint surface conditions. For instance, hydrothermal quartz veins with a strong welded bond to the wall rock would be among the highest intrablock qualities while weakly bonded, friable calcite or gypsum veins with poor adhesion would be among the lowest qualities.

Furthermore, a Massive row has been included to incorporate widely spaced structures. In particular, this enables strengthening intrablock structures, which may counteract other micro-defects in the rock at the field scale, to result in GSI values between 85 and 100. It should be noted that this chart can still be used for common jointed blocky rockmasses considered in conventional GSI applications, which contain only interblock structures, with guidelines discussed by Hoek et al. (2013).

This new GSI chart is designed for application to a broad scope of rockmass structures with the intention of universal application for rockmass characterization practice. While this is useful for characterization, the application of these results to numerical modelling using the Hoek-Brown criterion assumes the rockmass behaviour will be controlled by shear failure modes. In practice, the behaviour of massive rockmasses and strengthening intrablock structure is dominated by brittle spalling failure. Guidelines for the treatment of shear and squeezing and brittle rockmasses based on GSI values and intact compressive and tensile strength properties have been developed by Diederichs (2007) and will be discussed in detail in Chapter 4. For completeness here, a zone of expected brittle behaviour is indicated on the new GSI chart for complex rockmasses in Figure 3-7. For rockmasses that are described by this region of the chart, brittle rockmass failure criteria should be applied in numerical analyses.
Figure 3-6: New GSI chart for complex rockmasses that contain interblock and intrablock structures. The added column is used to describe the infill quality of strengthening intrablock structure and descriptions of other intrablock structure have been added to existing columns. A summary of equations to calculate the Composite GSI (CGSI) is also provided.
Figure 3-7: Region of expected brittle rockmass behaviour indicated on new GSI chart for complex rockmasses that contain interblock and intrablock structures, where brittle rockmass failure criteria should be applied in numerical analyses.
The GSI quantification by Hoek et al. (2013) proposed general scales for structure geometry (Scale A) and condition (Scale B) such that GSI is equal to their sum (see Equation 3.4). Values for Scales A and B that describe the rockmass structure can come from direct field observations using the GSI descriptions for block size and discontinuity condition, and/or scaled quantities from alternative geotechnical classification or characterization systems. A modified version of the Joint condition rating (JCond$_{89}$) from the 1989 version of RMR by Bieniawski (1989) is proposed here that includes intrablock structure as an alternative means to calculate values for Scale A (Equation 3.5). Similar to the proposed column addition and other modifications to the GSI chart for intrablock structure, the modifications to JCond$_{89}$ include an added column and modified descriptions in existing columns for intrablock structure (see Table 3-1). Alternative quantified inputs used here for Scale B are based on the GSI quantification by Cai et al. (2004) using logarithmic considerations of rock block volume (Equation 3.6).

\[
GSI = A_x + B_x \tag{3.4}
\]

\[
A_x \approx 1.5 \times JCond_{89} \tag{3.5}
\]

\[
B_x \approx 20 / 3 \times \log_{10} \text{(Block Volume in cm}^3\text{)} \tag{3.6}
\]

To calculate CGSI for a rockmass that contains multiple, distinct suites of structure, weighted composite values for Scale A and Scale B of all structure suites present are calculated using Equations 3.7 and 3.8. A* and B* are therefore equivalent blended parameters for the composite rockmass.

\[
A^* = \frac{(A_1 / B_1) + (A_2 / B_2) + ... + (A_n / B_n)}{(1 / B_1) + (1 / B_2) + ... + (1 / B_n)} \tag{3.7}
\]

\[
B^* = 20 \log_{10} \left(10^{-B_1/20} + 10^{-B_2/20} + ... + 10^{-B_n/20}\right)^{-1} \tag{3.8}
\]
Where $A_1$ and $B_1$ apply to the first structure suite (e.g. joints), $A_2$ and $B_2$ apply to the second structure suite (e.g. intrablock structure), and so on. The Composite GSI (CGSI) is then defined by Equation 3.9.

$$CGSI = A^* + B^*$$  \hspace{1cm} (3.9)

**Table 3-1: Modified Joint Condition rating, Modified JCond$_{sys}$, to include intrablock structure**

<table>
<thead>
<tr>
<th>Condition of discontinuities</th>
<th>Strengthening intrablock structure</th>
<th>Very rough or healed surfaces</th>
<th>Slightly rough surfaces or weak veins</th>
<th>Slickensided surfaces or Gouge &lt; 5 mm thick</th>
<th>Soft gouge &gt; 5 mm thick or Separation &gt; 5 mm Continuous</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overall rating</td>
<td>37.5</td>
<td>30</td>
<td>25</td>
<td>20</td>
<td>10</td>
</tr>
</tbody>
</table>

*Guidelines for classification of discontinuity conditions*

<table>
<thead>
<tr>
<th>Discontinuity length (persistence) Rating</th>
<th>&lt; 0.5 m</th>
<th>&lt; 1 m</th>
<th>1 to 3 m</th>
<th>3 to 10 m</th>
<th>10 to 20 m</th>
<th>&gt; 20 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rating</td>
<td>7.5</td>
<td>6</td>
<td>4</td>
<td>2</td>
<td>1</td>
<td>0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Separation (aperture) Rating</th>
<th>Welded</th>
<th>None</th>
<th>&lt; 0.1 mm</th>
<th>0.1 to 1.0 mm</th>
<th>1 to 5 mm</th>
<th>&gt; 5 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rating</td>
<td>7.5</td>
<td>6</td>
<td>5</td>
<td>4</td>
<td>1</td>
<td>0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Roughness Rating</th>
<th>Rough, undulating, irregular</th>
<th>Very rough</th>
<th>Rough</th>
<th>Slightly rough</th>
<th>Smooth</th>
<th>Slickensided</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rating</td>
<td>7.5</td>
<td>6</td>
<td>5</td>
<td>3</td>
<td>1</td>
<td>0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Infilling (gouge) Rating</th>
<th>Strong bonded vein (quartz)</th>
<th>None</th>
<th>Hard filling &lt; 5 mm</th>
<th>Hard filling &gt; 5 mm</th>
<th>Soft filling &lt; 5 mm</th>
<th>Soft filling &gt; 5 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rating</td>
<td>7</td>
<td>6</td>
<td>4</td>
<td>2</td>
<td>2</td>
<td>0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Weathering Rating</th>
<th>Strengthening by alteration</th>
<th>Unweathered</th>
<th>Slightly weathered</th>
<th>Moderate weathering</th>
<th>Highly weathered</th>
<th>Decomposed</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rating</td>
<td>7</td>
<td>6</td>
<td>5</td>
<td>3</td>
<td>1</td>
<td>0</td>
</tr>
</tbody>
</table>
3.6 FEM Validation Exercise of Composite GSI

A process using FEM numerical models was designed to compare models of an excavation with explicit rockmass structure to two corresponding implicit equivalent continuum models where the rockmass structure is represented by GSI and CGSI, respectively. The explicit model is ultimately compared to the implicit models to test the effectiveness of the Composite GSI approach for complex rockmasses with multiple suites of structure relative to a conventional, conservative GSI approach. The complex rockmass considered in this exercise contains one suite of interblock structure and one suite of intrablock structure. The rockmass responses around an excavation in explicit models with each suite of structure are calibrated to the corresponding behaviour in implicit models. The calibrated explicit rockmass structures are combined to a full explicit model with both suites of structure. Finally, the rockmass behaviour of the full explicit model is compared to implicit models of the full rockmass represented by the conventional GSI and Composite GSI approaches. The procedure of this validation exercise is illustrated in Figure 3-8.

Figure 3-8: Illustrative summary of FEM comparison of conventional GSI and CGSI
The simulated rockmass used in this study contains two suites of rockmass structure: three sets of joints with similar surface conditions (interblock structure) and a stockwork of healed but weakening (e.g. anhydrite as in Figure 3-9) veins (intrablock structure). The characteristics of each structure suite are defined using the GSI chart shown in Figure 3-6 and are listed in Table 3-2. Each suite of structure has been assigned its own GSI value, as if the rockmass contained only that suite.

![Figure 3-9: Example of anhydrite vein (Courtesy of Codelco Div. El Teniente)](image)

Table 3-2: Geometry of rockmass structure in terms of GSI parameters

<table>
<thead>
<tr>
<th>Condition Bin</th>
<th>Interblock: Joints</th>
<th>Intrablock: Anhydrite Veins</th>
<th>Full Rockmass</th>
</tr>
</thead>
<tbody>
<tr>
<td>Scale A</td>
<td>25</td>
<td>45</td>
<td>-</td>
</tr>
<tr>
<td>Structure Bin</td>
<td>“Blocky” (~50 cm spacing)</td>
<td>“Very Blocky” (~20 cm spacing)</td>
<td>-</td>
</tr>
<tr>
<td>Scale B</td>
<td>35</td>
<td>25</td>
<td>-</td>
</tr>
<tr>
<td>GSI</td>
<td>60</td>
<td>70</td>
<td>50</td>
</tr>
<tr>
<td>CGSI</td>
<td>-</td>
<td>-</td>
<td>62</td>
</tr>
</tbody>
</table>
The GSI value for the full rockmass with both suites of structure was selected as the worst case conventional approach while the Composite GSI (CGSI) value for the overall rockmass was determined using the methodology and equations described in Section 3.5.

The Composite GSI approach is designed to provide an improved estimate of rockmass strength for complex rockmasses. Field observations of other cases of excavations through complex rockmasses with interblock and healed intrablock structures indicate the conventional GSI approach underestimates their rockmass strength. This validation exercise compares models with individual explicit rockmass structures, which have each been calibrated to their implicit counterparts, to implicit models that represent the rockmass as a whole. Therefore, a real case study with observed or measured rockmass strain is not necessary for this comparison between the implicit models with minimal computational demand to the comprehensive explicit models.

3.6.1 Numerical Model Setup
This comparison between the GSI and CGSI approaches is conducted using FEM models of a 6 m-diameter circular excavation going through the selected rockmass (Figure 3-10a). The in situ stresses are approximated to a depth of 800 m with a K ratio of ~2.1, resulting in principal stresses of 45.1 MPa, 29.4 MPa and 21.6 MPa for $\sigma_1$ (horizontal and perpendicular to excavation axis), $\sigma_2$ (horizontal and parallel to excavation axis), and $\sigma_3$ (vertical), respectively. Intact rock properties in terms of the 2002 version of the Hoek-Brown strength criterion (Hoek et al., 2002) are listed in Table 3-3.
Figure 3-10: Finite-Element (RS2 by RocScience (2015)) model geometries of the circular 6 m-diameter excavation showing a) full model of implicit rockmass structure with mesh, external boundaries, and the four query measurement lines, inset (i) shows mesh detail near excavation; b) quarter model plus full excavation with explicit interblock (joint) geometry, inset (ii) shows structure geometry and mesh detail; c) quarter model plus full excavation with explicit intrablock (vein) geometry, inset (iii) shows structure geometry and mesh detail; d) quarter model plus full excavation with explicit interblock (joint) and intrablock (vein) geometries (i.e. full rockmass), inset (iv) shows structure geometry and mesh detail; all models have a far field section of an equivalent continuum region for computational stability, and the external boundaries have zero displacement (i.e. pinned) conditions.
Table 3-3: Intact rock properties in terms of the Generalized Hoek-Brown criterion

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Elastic Models</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intact Young’s Modulus, $E_i$ (MPa)</td>
<td>40,000</td>
</tr>
<tr>
<td>Poisson's Ratio, $\nu$</td>
<td>0.2</td>
</tr>
<tr>
<td>Uniaxial Compressive Strength, UCS (MPa)</td>
<td>110</td>
</tr>
<tr>
<td>$m_i$</td>
<td>15</td>
</tr>
<tr>
<td>$s$</td>
<td>1</td>
</tr>
<tr>
<td>$a$</td>
<td>0.5</td>
</tr>
<tr>
<td>GSI</td>
<td>100</td>
</tr>
</tbody>
</table>

3.6.2 Elastic Calibration Procedure

To obtain appropriate stiffness properties for the explicit rockmass structure elements, the following calibration procedure was conducted using the 2D FEM software RS2 by RocScience (2015). The calibration procedure for the interblock joint structure suite is as follows:

1. An elastic model of the excavation with implicit joints was created using a GSI value for the joint suite only (GSI = 60), as described in Table 3-2.

2. An elastic model of the excavation with explicit joints (Figure 3-10b) was created where the material properties represented the intact rock (see Table 3-3). The total displacement results of the explicit elastic model were compared to those of the implicit elastic model (from Step 1). A trial and error process of joint stiffness property selection was then applied with the aim of matching the total displacements between the elastic implicit and elastic explicit models in four directions around, and moving away from, the excavation boundary (query locations are illustrated in Figure 3-10). The joint stiffness properties that were tested in the trial runs are listed in Table 3-4 and the calibration results are shown in Figure 3-11.

3. A standard deviation analysis was conducted to statistically compare the joint stiffness trial runs to the total displacements in the corresponding implicit model. The measured sections of total displacements in the implicit model were set to zero, and the measurements from the explicit models were normalized to the implicit model (as in Figure 3-11). The trial with the smallest standard deviation measurement was selected to have the calibrated best fit joint stiffness properties. The results are listed in Table 3-5 with conditional formatting where red highlights the
highest standard deviation values and grades down through white to the lowest values highlighted in green (for each measurement location) or blue (for the average).

Figure 3-11: Graphs of elastic calibration for joints highlighting Trial 8 which was selected as the best fit to the implicit GSI model based on standard deviation analysis. Total displacements in the explicit models are normalized to those of the implicit model and stiffness units are in GPa/m
Table 3-4: Joint stiffness properties tested in elastic model trial runs

<table>
<thead>
<tr>
<th>Joint Model Trial #</th>
<th>Normal Stiffness, $K_n$ (MPa/m)</th>
<th>Shear Stiffness, $K_s$ (MPa/m)</th>
<th>$K_n$: $K_s$ Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>20,000</td>
<td>10,000</td>
<td>2</td>
</tr>
<tr>
<td>2</td>
<td>40,000</td>
<td>20,000</td>
<td>2</td>
</tr>
<tr>
<td>3</td>
<td>100,000</td>
<td>50,000</td>
<td>2</td>
</tr>
<tr>
<td>4</td>
<td>80,000</td>
<td>40,000</td>
<td>2</td>
</tr>
<tr>
<td>5</td>
<td>80,000</td>
<td>80,000</td>
<td>1</td>
</tr>
<tr>
<td>6</td>
<td>80,000</td>
<td>16,000</td>
<td>5</td>
</tr>
<tr>
<td>7</td>
<td>80,000</td>
<td>8,000</td>
<td>10</td>
</tr>
<tr>
<td>8</td>
<td>90,000</td>
<td>45,000</td>
<td>2</td>
</tr>
</tbody>
</table>

Table 3-5: Standard deviation analysis of normalized total displacements in elastic joint model trial runs; conditional formatting shows the highest values (red) and grades down through white to the lowest values (green) (for each measurement location) or blue (for the average)

<table>
<thead>
<tr>
<th>Joint Model Trial #</th>
<th>Standard deviations of total displacements with respect to the zeroed implicit model ($Y = 0$)</th>
<th>Top</th>
<th>Right</th>
<th>Bottom</th>
<th>Left</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td>0.000645</td>
<td>0.007967</td>
<td>0.000680</td>
<td>0.008036</td>
<td>0.004332</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>0.000232</td>
<td>0.002968</td>
<td>0.000280</td>
<td>0.002992</td>
<td>0.001618</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td>0.000046</td>
<td>0.000244</td>
<td>0.000080</td>
<td>0.000252</td>
<td>0.000155</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>0.000048</td>
<td>0.000422</td>
<td>0.000099</td>
<td>0.000438</td>
<td>0.000252</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>0.00188</td>
<td>0.00349</td>
<td>0.00199</td>
<td>0.00317</td>
<td>0.000264</td>
</tr>
<tr>
<td>6</td>
<td></td>
<td>0.000565</td>
<td>0.002244</td>
<td>0.000667</td>
<td>0.002150</td>
<td>0.001407</td>
</tr>
<tr>
<td>7</td>
<td></td>
<td>0.001510</td>
<td>0.005046</td>
<td>0.001618</td>
<td>0.004746</td>
<td>0.003230</td>
</tr>
<tr>
<td>8*</td>
<td></td>
<td>0.000042</td>
<td>0.000207</td>
<td>0.000086</td>
<td>0.000225</td>
<td>0.000140</td>
</tr>
<tr>
<td>Minimum Std. Dev.</td>
<td></td>
<td>0.000042</td>
<td>0.000207</td>
<td>0.000080</td>
<td>0.000225</td>
<td>0.000140</td>
</tr>
</tbody>
</table>

* Trial 8 was selected to be the best fit of elastic joint properties based on minimum average standard deviation

The calibration procedure for the intrablock vein structure suite is as follows:

4. In a similar fashion to Step 1 for joints, an elastic model of the excavation with implicit veins was created using a GSI value for the vein suite only (GSI = 70), as described in Table 3-2.

5. In a similar fashion to Step 2, an elastic model of the excavation with explicit veins (Figure 3-10c) was created where the material properties represented the intact rock (see Table 3-3). The total displacement results of the explicit elastic model were compared to those of the implicit elastic model (from Step 4). A trial and error process of vein stiffness property selection was then
applied with the aim of matching the total displacements between the elastic implicit and elastic explicit models in four directions around, and moving away from, the excavation boundary (query locations are illustrated in Figure 3-10). The vein stiffness properties that were tested in the trial runs are listed in Table 3-6 and the calibration results are shown in Figure 3-12.

6. In a similar fashion to Step 3, a standard deviation analysis was conducted to quantitatively compare the vein stiffness trial runs to the total displacements in the corresponding implicit model. The measured sections of total displacements in the implicit model were set to zero, and the measurements from the explicit models were normalized to the implicit model (as in Figure 3-12). The trial with the smallest standard deviation measurement was selected to have the calibrated best-fit vein stiffness properties. The results are listed in Table 3-7 with conditional formatting where red highlights the highest standard deviation values and grades down through white to the lowest values highlighted in green (for each measurement location) or blue (average).

Table 3-6: Vein stiffness properties tested in elastic model trial runs

<table>
<thead>
<tr>
<th>Vein Model Trial #</th>
<th>Normal Stiffness, $K_n$ (MPa/m)</th>
<th>Shear Stiffness, $K_s$ (MPa/m)</th>
<th>$K_n:K_s$ Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>100,000</td>
<td>100,000</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>200,000</td>
<td>100,000</td>
<td>2</td>
</tr>
<tr>
<td>3</td>
<td>200,000</td>
<td>150,000</td>
<td>1.333</td>
</tr>
<tr>
<td>4</td>
<td>200,000</td>
<td>200,000</td>
<td>1</td>
</tr>
<tr>
<td>5</td>
<td>300,000</td>
<td>300,000</td>
<td>1</td>
</tr>
<tr>
<td>6</td>
<td>400,000</td>
<td>400,000</td>
<td>1</td>
</tr>
<tr>
<td>7</td>
<td>400,000</td>
<td>200,000</td>
<td>2</td>
</tr>
<tr>
<td>8</td>
<td>500,000</td>
<td>500,000</td>
<td>1</td>
</tr>
<tr>
<td>9</td>
<td>500,000</td>
<td>250,000</td>
<td>2</td>
</tr>
<tr>
<td>10</td>
<td>600,000</td>
<td>600,000</td>
<td>1</td>
</tr>
<tr>
<td>11</td>
<td>600,000</td>
<td>300,000</td>
<td>2</td>
</tr>
<tr>
<td>12</td>
<td>500,000</td>
<td>300,000</td>
<td>1.667</td>
</tr>
<tr>
<td>13</td>
<td>500,000</td>
<td>100,000</td>
<td>5</td>
</tr>
<tr>
<td>14</td>
<td>500,000</td>
<td>200,000</td>
<td>2.5</td>
</tr>
<tr>
<td>15</td>
<td>600,000</td>
<td>250,000</td>
<td>2.4</td>
</tr>
</tbody>
</table>
Figure 3-12: Graphs of elastic calibration for veins highlighting Trial 15 which was selected as the best fit to the implicit GSI model based on standard deviation analysis. Total displacements in the explicit models are normalized to those of the implicit model and stiffness units are in GPa/m
Table 3-7: Standard deviation analysis of normalized total displacements in elastic vein model trial runs; conditional formatting shows the highest values (red) and grades down through white to the lowest values (green) (for each measurement location) or blue (for the average)

<table>
<thead>
<tr>
<th>Vein Model Trial #</th>
<th>Standard deviations of total displacements with respect to the zeroed implicit model (Y = 0)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top</td>
</tr>
<tr>
<td>1</td>
<td>0.000362</td>
</tr>
<tr>
<td>2</td>
<td>0.000093</td>
</tr>
<tr>
<td>3</td>
<td>0.000151</td>
</tr>
<tr>
<td>4</td>
<td>0.000211</td>
</tr>
<tr>
<td>5</td>
<td>0.000165</td>
</tr>
<tr>
<td>6</td>
<td>0.000143</td>
</tr>
<tr>
<td>7</td>
<td>0.000047</td>
</tr>
<tr>
<td>8</td>
<td>0.000131</td>
</tr>
<tr>
<td>9</td>
<td>0.000049</td>
</tr>
<tr>
<td>10</td>
<td>0.000124</td>
</tr>
<tr>
<td>11</td>
<td>0.000053</td>
</tr>
<tr>
<td>12</td>
<td>0.000075</td>
</tr>
<tr>
<td>13</td>
<td>0.000149</td>
</tr>
<tr>
<td>14</td>
<td>0.000023</td>
</tr>
<tr>
<td>15*</td>
<td>0.000030</td>
</tr>
<tr>
<td>Minimum Std. Dev.</td>
<td>0.000023</td>
</tr>
</tbody>
</table>

*Trial 15 was selected to be the best fit of elastic vein properties based on minimum average standard deviation

Two elastic implicit models of the excavation in the full rockmass, including both joints and veins, were created using the GSI and Composite GSI values. An elastic explicit model of the excavation containing all rockmass structure, joints and veins (Figure 3-10d), was created using the calibrated structural stiffness properties (Table 3-5 and Table 3-7). The explicit elastic model with both joints and veins was compared to the GSI and CGSI implicit models using total displacements measured vertically and horizontally around, and moving away from, the excavation boundary (Figure 3-10a). The results are shown in Figure 3-13, where the GSI and CGSI total displacements are normalized to the total displacements in the corresponding explicit model. A standard deviation analysis was used to statistically compare the fits of the GSI and CGSI models to the explicit model (Table 3-8). This comparison illustrates that, at all measurement locations, the elastic CGSI model is an improved match from the conventional GSI approach.
Figure 3-13: Total displacements of equivalent continuum (GSI and CGSI) models normalized to the model with explicit structure

Table 3-8: Standard deviation analysis of normalized total displacements in elastic rockmass model comparison; conditional formatting shows the highest values (red) and grades down through white to the lowest values (green) (for each measurement location) or blue (for the average)

<table>
<thead>
<tr>
<th>Implicit Models</th>
<th>Top</th>
<th>Right</th>
<th>Bottom</th>
<th>Left</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>GSI</td>
<td>0.000174</td>
<td>0.002779</td>
<td>0.000223</td>
<td>0.002768</td>
<td>0.001486</td>
</tr>
<tr>
<td>CGSI*</td>
<td><strong>0.000097</strong></td>
<td><strong>0.001432</strong></td>
<td><strong>0.000064</strong></td>
<td><strong>0.001459</strong></td>
<td><strong>0.000763</strong></td>
</tr>
<tr>
<td>Minimum Std. Dev.</td>
<td><strong>0.000097</strong></td>
<td><strong>0.001432</strong></td>
<td><strong>0.000064</strong></td>
<td><strong>0.001459</strong></td>
<td><strong>0.000763</strong></td>
</tr>
</tbody>
</table>
3.6.3 Plastic Calibration Procedure

For the elasto-plastic models comparing rockmass strength, a calibration procedure similar to that of the elastic models was conducted to obtain appropriate strength properties for the explicit structure elements. The elasto-plastic models are calibrated and ultimately compared using measurements of the depth of plastic yield into the rockmass from the excavation boundary at 45 degree increments around the excavation. The joint strength properties that were tested in the trial runs are listed in Table 3-9 and the tested vein strength properties are listed in Table 3-10. The measurements of the depth of plastic yield for the trial calibration models are shown in Figure 3-14 for joints and Figure 3-15 for veins. The standard deviation analyses to statistically select the best fit strength properties are shown in Table 3-11 for joints and in Table 3-12 for veins. The in situ stress anisotropy had a marked effect on the models where larger depths of yield occurred in the top and bottom of the excavations. The calibrated explicit plastic joint and vein models are shown in Figure 3-16. A precise calibration of the joint and vein strength parameters to match the corresponding implicit plastic models was challenging due to the geometry of the explicit structure.

### Table 3-9: Joint strength properties tested in plastic model trial runs

<table>
<thead>
<tr>
<th>Trial #</th>
<th>Peak</th>
<th>Residual</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tensile strength, $\sigma_t$ (MPa)</td>
<td>Cohesion, c (MPa)</td>
</tr>
<tr>
<td>1</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>0</td>
<td>5</td>
</tr>
<tr>
<td>3</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>4</td>
<td>0</td>
<td>5</td>
</tr>
<tr>
<td>5</td>
<td>0</td>
<td>4</td>
</tr>
<tr>
<td>6</td>
<td>0</td>
<td>4</td>
</tr>
<tr>
<td>7</td>
<td>0</td>
<td>4</td>
</tr>
<tr>
<td>8</td>
<td>0</td>
<td>3</td>
</tr>
<tr>
<td>9</td>
<td>0</td>
<td>3</td>
</tr>
</tbody>
</table>
Table 3-10: Vein strength properties tested in plastic model trial runs

<table>
<thead>
<tr>
<th>Trial #</th>
<th>Peak</th>
<th>Residual</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tensile strength, $\sigma_t$ (MPa)</td>
<td>Cohesion, $c$ (MPa)</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>5</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>3</td>
<td>0.5</td>
<td>2</td>
</tr>
<tr>
<td>4</td>
<td>0.5</td>
<td>1</td>
</tr>
<tr>
<td>5</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>6</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>7</td>
<td>2</td>
<td>5</td>
</tr>
<tr>
<td>8</td>
<td>2</td>
<td>5</td>
</tr>
</tbody>
</table>

Figure 3-14: Graph of plastic model calibration for joint strength by measuring depth of plastic yield around the excavation
Figure 3-15: Graph of plastic model calibration for vein strength by measuring depth of plastic yield around the excavation

Table 3-11: Standard deviation analysis of normalized yield depth in plastic joint model trial runs; conditional formatting shows the highest values (red) and grades down through white to the lowest values (green) (for each measurement location) or blue (for the average)

<table>
<thead>
<tr>
<th>Joint Model Trial #</th>
<th>Standard deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>7.107729</td>
</tr>
<tr>
<td>2</td>
<td>4.360800</td>
</tr>
<tr>
<td>3</td>
<td>0.784253</td>
</tr>
<tr>
<td>4</td>
<td>0.714340</td>
</tr>
<tr>
<td>5</td>
<td>0.647084</td>
</tr>
<tr>
<td>6*</td>
<td>0.529441</td>
</tr>
<tr>
<td>7</td>
<td>0.654073</td>
</tr>
<tr>
<td>8</td>
<td>0.682372</td>
</tr>
<tr>
<td>9</td>
<td>0.699607</td>
</tr>
<tr>
<td>Minimum Std. Dev.</td>
<td>0.529441</td>
</tr>
</tbody>
</table>

*Trial 6 selected to be the best fit for plastic joint strength properties based on minimum average standard deviation.
Table 3-12: Standard deviation analysis of normalized yield depth in plastic vein model trial runs; conditional formatting shows the highest values (red) and grades down through white to the lowest values (green) (for each measurement location) or blue (for the average)

<table>
<thead>
<tr>
<th>Vein Model Trial #</th>
<th>Standard deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.313725</td>
</tr>
<tr>
<td>2*</td>
<td>0.259474</td>
</tr>
<tr>
<td>3</td>
<td>0.295563</td>
</tr>
<tr>
<td>4</td>
<td>0.325743</td>
</tr>
<tr>
<td>5</td>
<td>0.647479</td>
</tr>
<tr>
<td>6</td>
<td>0.490276</td>
</tr>
<tr>
<td>7</td>
<td>2.840118</td>
</tr>
<tr>
<td>8</td>
<td>0.735550</td>
</tr>
<tr>
<td>Minimum Std. Dev.</td>
<td>0.259474</td>
</tr>
</tbody>
</table>

*Trial 2 selected to be the best fit for plastic vein strength properties based on minimum average standard deviation

Figure 3-16: Calibrated plastic models showing depth of yield measurements from excavation boundary
In a similar fashion to the elastic comparison of GSI and CGSI, two plastic implicit models of the excavation in the full rockmass were created. A plastic explicit model of the excavation containing all rockmass structure (Figure 3-16 right) was created using the calibrated joint and vein stiffness and strength properties. The measurements of the depth of plastic yield were used to compare the explicit and two implicit models. The results were normalized to the explicit model, and a standard deviation analysis was used to quantitatively compare the fits of the implicit GSI and CGSI models (Table 3-13 and Figure 3-17). Like the results of elastic model total displacement analysis, the Composite GSI (CGSI) is an improved fit to the explicit model compared to the conventional approach.

Overall, the conventional GSI for the rockmass was found to generate larger total displacements and depths of plastic yield in this analysis when compared to the calibrated explicit model. This supports field observations of other complex rockmasses that contain suites of interblock and intrablock structures that suggest the conventional GSI approach underestimates rockmass strength in a complex rockmass where both joints (interblock structures) and veins (intrablock structures) are considered.

Table 3-13: Standard deviation analysis of normalized plastic yield depth in rockmass model comparison; conditional formatting shows the highest values (red) and grades down through white to the lowest values (green)

<table>
<thead>
<tr>
<th>Implicit Models</th>
<th>Standard deviation from explicit model</th>
</tr>
</thead>
<tbody>
<tr>
<td>GSI</td>
<td>0.367571</td>
</tr>
<tr>
<td>CGSI*</td>
<td>0.211384</td>
</tr>
<tr>
<td>Minimum Std. Dev.</td>
<td>0.211384</td>
</tr>
</tbody>
</table>
Figure 3-17: Depth of yield of plastic equivalent continuum (GSI and CGSI) models normalized to the model with explicit structure
3.7 Optimizing Numerical Simulation of Rockmasses with Multiple Suites of Structure

It is important to consider all rockmass structure in geotechnical design in some fashion. For rockmasses that have multiple distinct suites of structure, the computational demand to model all suites of structure explicitly or discretely becomes a limiting factor for numerical models and some optimal balance is therefore required. The philosophy of the new Composite GSI approach, where individual suites of rockmass structure are considered separately, presents a solution for a technique to optimize numerical modelling of rockmass structures. In this case, the most computationally demanding structures such as those with complex geometries or relatively small spacing (e.g. less than 20 cm for a tunnel scale model) can be represented implicitly by an equivalent continuum material, while other rockmass structures can be modelled explicitly to capture structurally-driven rockmass behaviour. To apply this approach effectively, it is important to have a sound understanding of the rockmass and particularly its structure in order to accurately represent the rockmass behaviour and failure modes, as well as their influence on design.

Two case examples of underground excavations with arched roofs are used to highlight the implications of inclusion and exclusion of explicit structure in FEM numerical models (using Phase² software by RocScience, 2011). The two cases consist of (A) a shallow cavern in basaltic volcanic flows, and (B) a shallow tunnel in carbonate sedimentary strata. Each case has four models that assess a stepwise progression between fully explicit and fully implicit inclusions of rockmass structure. The rockmass structure in the models ranges from a fully explicit representation with multiple joint element networks and intact rock as the continuum material, to a partially explicit representation with some suites of structure represented by an equivalent continuum material defined by a CGSI value, through to fully implicit approximations using a CGSI equivalent continuum material (see Table 3-14 and Figure 3-19). The models with all explicit structure use a CGSI value of 100 for the intact material, where the strength properties are in terms of the Hoek-Brown strength criterion (Hoek et al., 2002), because it is assumed that all microdefects have been accounted for in the explicitly modelled rockmass structures.

Although the quantified GSI chart after Hoek and Marinos (2000) and Cai et al. (2004) (Figure 3-2) has been used to calculate CGSI for these cases, the same approach using the new linearized and
quantified CGSI chart (Figure 3-6) is also valid. This technique can be implemented in the design of large-scale geotechnical projects, where smaller sample sections of a project can be tested through the full stepwise progression of models to ensure an appropriate understanding of the rockmass behaviour. The optimized modelling solution at an appropriate balance between explicit and implicit structures (to meet computational limitations while maintaining an accurate representation of rockmass behaviour) would then be available for modelling the broader scale of a project.

Table 3-14: Descriptions and corresponding material CGSI values for structure in each model

<table>
<thead>
<tr>
<th>Model #</th>
<th>1 (CGSI Value)</th>
<th>2 (CGSI Value)</th>
<th>3 (CGSI Value)</th>
<th>4 (CGSI Value)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case A</td>
<td>All explicit structure (100)</td>
<td>Implicit vertical &amp; horizontal sub-column joints (65)</td>
<td>Implicit vertical &amp; horizontal sub-column joints &amp; columnar basalt joints (59)</td>
<td>All implicit structure (59)</td>
</tr>
<tr>
<td>Case B</td>
<td>All explicit structure (100)</td>
<td>Implicit healed nodular structure (64)</td>
<td>Implicit healed nodular structure, cross joints &amp; bedding (55)</td>
<td>All implicit structure (48)</td>
</tr>
</tbody>
</table>

Figure 3-18: Geometry of explicit structure suites for each model relative to the excavation sizes for both cases
3.7.1 Case A: Cavern in Basaltic Volcanic Flows

Case A considers a cavern excavation in basaltic volcanic flow lithological units. The cross section of the cavern is 16 m high and 34 m wide with an arched roof beginning 5 m above the floor, as shown in Figure 3-20. Gravitational stresses act on the excavation, where the cavern floor is situated 60 m below ground surface and the unit weight of the overburden is assumed to be approximately 0.027 MPa/m. An in situ stress ratio \( K = \sigma_H : \sigma_v \) of 1.5 was applied both in and out of plane. The selected stiffness and strength properties of the intact rock are listed in Table 3-15.
Five suites of rockmass structure are included in the models for Case A; their stiffness and strength properties are shown in Table 3-16 and their geometry is shown in Figure 3-18 and Figure 3-20. Normal and shear stiffness values are approximated from data presented by Read and Stacey (2009). All suites of structure in this model are interblock structure at a range of scales. The horizontal volcanic flow beds and columnar basalt joints form during initial deposition and cooling of lava flows and are the typical structures considered in rockmass characterization in a basalt rockmass. Horizontal and vertical sub-column joints are included in this investigation to account for the effects of sub-column cooling and slight metamorphism (folding). The subvertical super macro joints are large scale joints, possibly with minor fault slip, that likely formed during metamorphism.

Table 3-15: Stiffness and strength properties of intact rock for Case A

<table>
<thead>
<tr>
<th>Material Parameter (Units)</th>
<th>Intact Rock</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s modulus, E (MPa)</td>
<td>50,000</td>
</tr>
<tr>
<td>Poisson’s ratio, ν</td>
<td>0.25</td>
</tr>
<tr>
<td>Generalized Hoek-Brown strength criterion (Hoek et al., 2002)</td>
<td></td>
</tr>
<tr>
<td>Intact compressive strength, σ_{ci} (MPa)</td>
<td>200</td>
</tr>
<tr>
<td>m_i</td>
<td>22</td>
</tr>
<tr>
<td>Dilation parameter</td>
<td>0</td>
</tr>
<tr>
<td>Disturbance factor</td>
<td>0.7</td>
</tr>
</tbody>
</table>

Table 3-16: Stiffness and strength properties of explicit structure in Case A

<table>
<thead>
<tr>
<th>Structure</th>
<th>Average spacing (m)</th>
<th>Normal stiffness, K_n (MPa/m)</th>
<th>Shear stiffness, K_s (MPa/m)</th>
<th>Barton-Bandis</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical sub-column joints</td>
<td>0.6</td>
<td>25,000</td>
<td>14,000</td>
<td>180 15 32</td>
</tr>
<tr>
<td>Horizontal sub-column joints</td>
<td>2</td>
<td>25,000</td>
<td>14,000</td>
<td>170 12 32</td>
</tr>
<tr>
<td>Columnar basalt joints</td>
<td>2</td>
<td>10,000</td>
<td>6,000</td>
<td>160 7 32</td>
</tr>
<tr>
<td>Horizontal volcanic flow bed</td>
<td>5</td>
<td>10,000</td>
<td>6,000</td>
<td>140 10 31</td>
</tr>
<tr>
<td>Subvertical super macro joints</td>
<td>15</td>
<td>10,000</td>
<td>6,000</td>
<td>120 5 31</td>
</tr>
</tbody>
</table>
Figure 3-20: FEM models (A1 to A4) of Case A showing deformed $\sigma_1$ contours, yielded material (mesh) elements, and yielded joint elements. Deformation scale factor is 10

Conventional numerical modelling would either not consider the horizontal and vertical sub-column joints or model them using an equivalent continuum, as in Model A2. Compared to Model A1 which has fully explicit structure, Models A2, A3, and A4 have significantly less joint yield propagation above and wedge failure in the roof of the cavern.

Overall, these models show a significant difference in failure mode and extent between Models A1 and A4. The majority of failure, in this case, occurs in the joint elements, and equivalent continuum models with implicit structure do not effectively capture the rockmass behaviour. Regarding computation time, there was not a substantial difference between Models A1 and A2. Therefore, in this case, the detailed rockmass behaviour captured by fully explicit structure is not limited by current computational capacity, so Model A1 would be the preferred model to use in further investigations.

3.7.2 Case B: Tunnel in Carbonate Sedimentary Strata
Case B considers a tunnel in the carbonate sedimentary Lindsay Formation near Ottawa, Canada. The cross section of the tunnel is 8 m high and 12 m wide with an arched roof beginning 5 m above the floor. Gravitational stresses act on the excavation, where the tunnel floor is at 17 m below ground surface (13 m below the contact between soil and bedrock). A locked-in horizontal stress (both in and out of plane) of
2 MPa is applied to the bedrock in the model but not the overburden. The selected stiffness and strength properties of the intact rock and soil overburden are listed in Table 3-17.

**Table 3-17: Stiffness and strength properties of intact rock and soil overburden for Case B**

<table>
<thead>
<tr>
<th>Material Parameter (Units)</th>
<th>Intact Rock</th>
<th>Soil overburden</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young's modulus, $E$ (MPa)</td>
<td>30,000</td>
<td>150</td>
</tr>
<tr>
<td>Poisson's ratio, $\nu$</td>
<td>0.3</td>
<td>0.3</td>
</tr>
<tr>
<td><strong>Generalized Hoek-Brown strength criterion (Hoek et al., 2002)</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Intact compressive strength, $\sigma_i$ (MPa)</td>
<td>80</td>
<td>-</td>
</tr>
<tr>
<td>$m_i$</td>
<td>12</td>
<td>-</td>
</tr>
<tr>
<td>Dilation parameter</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>Disturbance factor</td>
<td>0.5</td>
<td>-</td>
</tr>
<tr>
<td><strong>Mohr-Coulomb strength criterion</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tensile strength, $\sigma_t$ (MPa)</td>
<td>-</td>
<td>0</td>
</tr>
<tr>
<td>Internal friction angle, $\phi$ (°)</td>
<td>-</td>
<td>35</td>
</tr>
<tr>
<td>Cohesion, $c$ (MPa)</td>
<td>-</td>
<td>0</td>
</tr>
</tbody>
</table>

Five suites of rockmass structure are considered in Case B; their stiffness and strength properties are listed in Table 3-18 and their geometry is shown in Figure 3-18 and Figure 3-21. Normal and shear stiffness values are from those presented by Read and Stacey (2009). This rockmass contains both interblock and intrablock structure. The intrablock structure is defined by tortuous layers with high clay content that surround calcite-rich nodules. The nodular intrablock structure, bedding, and clay seams comprise structures resulting from the initial deposition and lithification of the limestone, closely followed by cross joints between bedding layers. The subvertical joints likely formed due to subsequent tectonic activity (NWMO, 2011). In outcrop, the intrablock structure weathers preferentially, leaving the limestone nodules as the remaining intact rock; however, the same sections of rock in drill core are competent and the intrablock structure and limestone behave together as intact rock. The strength properties of the nodular structure are estimated from direct shear tests on the deeper Cobourg limestone unit that is the stratigraphic equivalent to the Lindsay Formation (NWMO, 2011). Lower bound peak and residual strengths were selected from the data to account for the difference in depth of (and therefore stress acting on) the units, as shown by the purple lines in Figure 3-22.
Table 3-18: Stiffness and strength properties of explicit structure in Case B

<table>
<thead>
<tr>
<th>Structure</th>
<th>Average spacing (m)</th>
<th>Normal stiffness, ( K_n ) (MPa/m)</th>
<th>Shear stiffness, ( K_s ) (MPa/m)</th>
<th>Peak / Residual (Mohr-Coulomb)</th>
<th>Barton-Bandis</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( \sigma_t ) (MPa) ( \phi ) (°)</td>
<td>JCS</td>
</tr>
<tr>
<td>Nodular structure</td>
<td>0.1</td>
<td>8,000</td>
<td>6,000</td>
<td>0.01 / 0</td>
<td>63 / 27</td>
</tr>
<tr>
<td>Cross joints</td>
<td>3</td>
<td>35,000</td>
<td>10,000</td>
<td>0.01 / 0</td>
<td>24 / 22</td>
</tr>
<tr>
<td>Bedding</td>
<td>0.75</td>
<td>35,000</td>
<td>10,000</td>
<td>0.01 / 0</td>
<td>26 / 24</td>
</tr>
<tr>
<td>Subvertical joints</td>
<td>5</td>
<td>8,000</td>
<td>6,000</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Clay seam</td>
<td>2.5</td>
<td>8,000</td>
<td>6,000</td>
<td>0.01 / 0</td>
<td>16 / 15</td>
</tr>
</tbody>
</table>

Figure 3-21: FEM models (B1 to B4) of Case B showing deformed \( \sigma_1 \) contours, yielded material (mesh) elements, and yielded joint elements. Deformation scale factor is 2

Figure 3-22: Direct shear results on a deeper limestone unit similar to the Lindsay Formation. The lower bound peak and residual strengths are used to determine the strength of the nodular intrablock structure (modified after NWMO, 2011)
Since intrablock structure is not considered in conventional rockmass characterization and subsequent numerical modelling, a model similar to B2 (one using a GSI of 100 instead of a CGSI of 64 that accounts for the intrablock structure) would likely be used to investigate the stability of this tunnel. There is a significant difference in total displacement between Model B1 and Models B2 to B4 due to the failure of the intrablock structure, where Model B1 features an elevated stress concentration, and Models B2 to B4 show minimal displacement. The differences in total displacement at the excavation boundary extend to the surface where ground subsidence reaches 0.01 m, 0.005 m, 0.004 m, and 0.0006 m in Models B1 through B4, respectively.

In Model B1, the yield zone of the intrablock structure (represented by a Voronoi geometry joint network) extends into the rockmass by approximately 2 m from the sides of the excavation and 0.3 m from the top. In Models B2 to B4, there is a higher concentration of mesh element failure that extends to a similar distance into the rockmass from the excavation when compared to the Voronoi joint element failure in Model B1. This suggests that the behaviour of the intrablock structure is captured in Models B2 to B4 where the intrablock structure is implicitly represented by equivalent continua materials.

Similar to Case A, these models show a significant difference in rockmass behaviour and failure modes. Although the majority of failure in Model B1 occurs in the Voronoi joint elements of the intrablock structure, it is mostly captured by mesh element failure in Models B2, B3, and B4. In this case, there is a significant difference in computation time between Model B1 and the others. Overall, Model B2 appears sufficient to capture the behaviour of the rockmass with the assumption of a sound understanding of the influence of the nodular intrablock structure.

This optimization analysis to balance the implicit and explicit rockmass structure in a model can be applied to any rockmass that contains multiple suites of either interblock structures, intrablock structures, or both. The two cases presented in this section, of a cavern in basaltic volcanic flows and a tunnel in carbonate sedimentary strata, were selected based on available case data. Rockmass behaviour for other complex rockmasses, such as those with hydrothermal vein intrablock structures, can certainly be assessed using this technique.
3.8 El Teniente Adit Case to Validate the Composite GSI Method

In this section, the CGSI method, as applied to behaviour evaluating using equivalent continuum FEM models, is used to analyze a validation case study of an adit at the El Teniente porphyry Cu-Mo mine in Chile. The El Teniente copper-molybdenum porphyry mine owned by Codelco is currently the largest underground block caving operation in the world (e.g. Stern et al., 2011). It is located in the Andean Cordillera in central Chile, approximately 70 km SSE of the capital city, Santiago. The main rock types in the deposit include breccia, andesite, diorite, and a stockwork mafic complex. The stockwork intrablock structure is known to affect rockmass behaviour (Brzovic and Villaescusa, 2007). The mine has been in operation since the early 20th century. Higher elevations of the deposit have been completely mined, leaving a large subsidence crater overlying current active mine levels. The adit considered for this case study is planned to connect from ground surface to the new mine level, up to a depth of approximately 1000 m below ground surface.

3.8.1 Site Observations

The sections of the adit considered for this study are excavation faces observed at depths of approximately 450, 550, and 600 m, as shown in Figure 3-23. The design profile for the adit is an arched roof geometry, approximately 6 m high and 6 m wide. All observed excavation faces occur in the stockwork mafic complex geological unit. Several joint sets (interblock structure) were observed in detail at the 600 m deep face in addition to a stockwork suite of hydrothermal quartz veins (intrablock structure). Annotated photos of the excavation face at 600 m (Figure 3-26) show four joint sets highlighted by blue, green, pink, and yellow polygons. The approximate orientations of the joint planes and adit are shown in a stereonet in Figure 3-24. The adit is advancing eastward. The average spacing of the quartz veins is defined by the fragmented block sizes of the excavated material, which are visible in the muck pile at the face in Figure 3-26c. Vein spacing controlling the fragmented block size is consistent with field observations by the author and Brzovic and Villaescusa (2007). The range of fragmented block sizes is shown in Figure 3-25, where a normal distribution calculates an average vein spacing of 0.39 m.
All four joint sets are considered together as a single suite of structure, with the same surface condition ranked as “very good” quality on the GSI chart. The joint spacing is greater than 1 m, resulting in a “blocky” ranking on the GSI chart. The quartz veins comprise a second suite of structure, ranked as high quality “strengthening intrablock structure” and moderate “very blocky” on the GSI chart (see Figure 3-2). Based on these GSI assessments, Scales A and B (Equations 3.5 to 3.8) and corresponding calculated GSI and CGSI values are listed in Table 3-19.

Ignoring intrablock structure would generate GSI for only joints (GSI = 80), which is considered an overestimate of rockmass strength. Likewise, a conservative conventional approach considers the worst case of the combined suites of structure, GSI = 65, which is considered to underestimate rockmass strength. These scenarios are compared to the CGSI assessment of CGSI = 73 using FEM models in the following sections.

**Table 3-19: GSI properties of observed rockmass structure**

<table>
<thead>
<tr>
<th>Structure</th>
<th>Scale A</th>
<th>Scale B</th>
<th>GSI</th>
</tr>
</thead>
<tbody>
<tr>
<td>All joint sets combined</td>
<td>40</td>
<td>40</td>
<td>80</td>
</tr>
<tr>
<td>Hydrothermal veins</td>
<td>55</td>
<td>25</td>
<td>80</td>
</tr>
<tr>
<td>Conventional worst case GSI</td>
<td>40</td>
<td>25</td>
<td>65</td>
</tr>
<tr>
<td>Composite GSI, CGSI</td>
<td>A* = 49</td>
<td>B* = 24</td>
<td>CGSI = 73</td>
</tr>
</tbody>
</table>

Figure 3-23: Excavation faces along the adit at various depths below ground surface
Figure 3-24: Stereonet showing observed adit and joint set orientations (right-hand strike/dip)

Figure 3-25: Histogram of selected block sizes as measurement of average spacing of quartz veins
Figure 3-26: Site observations from the 600 m deep excavation face of the adit at El Teniente. (a) Approximate excavation profile of adit approximately 5 m behind the face; (b) view of excavation face including immediate roof; (b) detailed view of excavation face with highlighted joint planes (4 sets) and average quartz vein spacing defined by the fragmented block size of the excavated material.
3.8.2 General Model Setup

FEM numerical models of the adit were created using RS2 software by RocScience (2015). The adit is assumed to be far enough away from any other excavation such that it is not affected by other induced stresses. In situ stresses have been measured in the mine at various locations using multiple techniques including overcoring and borehole breakout observations, which were analyzed by Diederichs (2016). The minor principal stress ($\sigma_3$) is oriented vertically while the major ($\sigma_1$) and intermediate ($\sigma_2$) principal stresses are oriented horizontally. The K ratios between $\sigma_1$ vs. $\sigma_3$ and $\sigma_2$ vs. $\sigma_3$ tend to decrease with increasing depth (D) (see Figure 3-27). The maximum K ratio relationships are:

\[
\begin{align*}
\sigma_1 &= \sigma_3 (1 + 60D^{-0.6}) \\
\sigma_2 &= \sigma_3 (1 + 20D^{-0.6})
\end{align*}
\] (3.10) (3.11)

An estimated intermediate set of K ratio relationships was selected for this case study, however, to account for stress rotations caused by mining activities (e.g. McKinnon and de la Barra, 2003), as follows:

\[
\begin{align*}
\sigma_1 &= \sigma_3 (1 + 40D^{-0.6}) \\
\sigma_1 &= \sigma_3 (1 + 10D^{-0.6})
\end{align*}
\] (3.12) (3.13)

The explicit rockmass structure in the models is based on site observations from the 600 m deep excavation face (see Figure 3-26). The equivalent continuum region of the explicit models is implemented in the far field sections of the model, away from the adit (see Figure 3-28), to alleviate the high computational requirements required for explicit structure.
The geometries of the rockmass structure are visible in Figure 3-28 (inset) where the joints are modelled with parallel statistical elements and the veins are modelled with Voronoi polygonal elements. Joint set 2 is excluded from these 2D models because it is nearly perpendicular to the excavation.

The intact rock properties of the hydrothermal mafic complex (Table 3-20) are from laboratory testing conducted at the mine. The geometry of the modelled rockmass structure is based on site observations (Table 3-21). The mechanical properties of the joints and veins (Table 3-22) are based on results from the literature (Read and Stacey, 2009) and work conducted by the author (presented in Chapter 5 of this thesis and published in Day et al., 2014).
Figure 3-28: Relevant quadrant of FEM model with explicit structure, and a detailed inset of explicit structure and adit dimensions

Table 3-20: Intact properties of hydrothermal mafic complex

<table>
<thead>
<tr>
<th>Parameter (Units)</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intact Young’s Modulus, $E_i$ (MPa)</td>
<td>60,000</td>
</tr>
<tr>
<td>Poisson’s Ratio, $\nu$</td>
<td>0.18</td>
</tr>
<tr>
<td>Unconfined Compressive Strength, $\sigma_c$ (MPa)</td>
<td>120</td>
</tr>
<tr>
<td>Hoek-Brown material constant, $m_i$</td>
<td>9.1</td>
</tr>
<tr>
<td>Hoek-Brown material constant, $s$</td>
<td>1</td>
</tr>
<tr>
<td>Hoek-Brown material constant, $a$</td>
<td>0.5</td>
</tr>
</tbody>
</table>
Table 3-21: Geometry of modelled rockmass structure

<table>
<thead>
<tr>
<th>Parameter (Units)</th>
<th>Joint Set 1</th>
<th>Joint Set 2</th>
<th>Joint Set 3</th>
<th>Quartz veins</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inclination (°)</td>
<td>12</td>
<td>-85</td>
<td>-15</td>
<td>-</td>
</tr>
<tr>
<td>Average spacing (m)</td>
<td>3</td>
<td>2.5</td>
<td>1</td>
<td>0.25</td>
</tr>
<tr>
<td>Average length (m)</td>
<td>4</td>
<td>2</td>
<td>2</td>
<td>-</td>
</tr>
<tr>
<td>Persistence</td>
<td>0.9</td>
<td>0.7</td>
<td>0.7</td>
<td>-</td>
</tr>
<tr>
<td>Joint end condition</td>
<td>Open</td>
<td>Open</td>
<td>Open</td>
<td>Open</td>
</tr>
<tr>
<td>Voronoi regularity</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Irregular</td>
</tr>
</tbody>
</table>

Table 3-22: Mechanical properties of rockmass structure

<table>
<thead>
<tr>
<th>Parameter (Units)</th>
<th>Joints</th>
<th>Quartz Veins</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal stiffness, Ke (MPa/m)</td>
<td>30,000</td>
<td>6,500,000</td>
</tr>
<tr>
<td>Shear stiffness, Ks (MPa/m)</td>
<td>10,000</td>
<td>6,500,000</td>
</tr>
</tbody>
</table>

Mohr-Coulomb Strength Criterion

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Joints</th>
<th>Quartz Veins</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak tensile strength, σt (MPa)</td>
<td>0</td>
<td>0.1</td>
</tr>
<tr>
<td>Peak cohesion, c (MPa)</td>
<td>1</td>
<td>4.3</td>
</tr>
<tr>
<td>Peak friction angle, ϕ (°)</td>
<td>55</td>
<td>25</td>
</tr>
<tr>
<td>Residual tensile strength, σr (MPa)</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Residual cohesion, c (MPa)</td>
<td>0</td>
<td>0.2</td>
</tr>
<tr>
<td>Residual friction angle, ϕ (°)</td>
<td>25</td>
<td>40</td>
</tr>
</tbody>
</table>

3.8.3 Model Analysis using Depth of Rockmass Yield

FEM models of the adit with rockmass structure observed at the 600 m deep face were created using:

(i) explicit rockmass structure,

and three equivalent continuum models with implicit structure represented by:

(ii) joints only (highest GSI) that ignores the presence of intrablock structure,

(iii) Composite GSI values, and

(iv) worst case conventional GSI values.

The models are compared using depth of plastic yield from the excavation boundary, measured at the roof, as an analogue for overbreak. Detailed results are shown in Figure 3-29. The yield of both the structural elements and intact rock are considered for the explicit model. Only yielded elements that can be traced to the excavation boundary through other yielded segments are included. Single yielded
segments surrounded by otherwise intact rock and rockmass structure (typically far from the excavation) are assumed to have insignificant influence on overbreak. In the model with explicit structure, most of the yield occurs through the structural elements instead of the intact rock. This is consistent with site observations (Figure 3-26) where most failure at the 600 m face occurred along the joints and veins, and further block fragmentation occurred along and/or through the veins.

When comparing the explicit model of the 600 m deep face to the implicit equivalent continuum models represented by three different GSI approaches, the CGSI model is the best estimate of roof depth of yield (Figure 3-29 and Figure 3-30). This finding validates the use of CGSI for continuum modelling of complex rockmasses with intrablock structure at depth.

![Figure 3-29: FEM model results of the 600 m deep excavation face of the adit at El Teniente, comparing maximum principal stresses ($\sigma_1$) and yielded elements, for the explicit and GSI equivalent continuum models; depth of yield measurements in the roof are indicated for each model](image)

3.8.4 Extension of Analysis to Various Excavation Depths

The FEM models with rockmass structure based on the 600 m deep excavation face were exposed to numerous excavation depths between 100-2000 m below ground surface to investigate the applicability of the CGSI method for a range of stress conditions. The in situ stress conditions for these models vary according to the stress analysis by Diederichs (2016) (see Figure 3-27). The model results for depth of
yield measurements are shown in Figure 3-30. The nonlinear curve fit selected for these data sets is the Carreau-Yasuda model that is designed to describe pseudoplastic flow with asymptotic viscosities at zero and infinite shear rates. The Carreau-Yasuda model enables asymptotic behaviour toward zero depth of yield in shallow conditions. These best-fit functions were solved using the Levenberg Marquardt iteration algorithm.

The overbreak site observations at the 450 and 550 m excavation faces are plotted in Figure 3-30 and, like the 600 m model, their explicit models are in good agreement with the CGSI models. The worst-case conventional GSI approach underestimates the rockmass strength, resulting in a significantly larger depth of yield when compared to the explicit models, CGSI models, and field observations. In contrast, the conventional joints-only GSI that ignores intrablock structure overestimates the rockmass strength, resulting in an underestimated depth of yield. The consequences for both conventional approaches must be considered in the design of primary support, where optimized bolt lengths are required to support the rockmass effectively and efficiently. The analyses at 450 and 550 m are further evidence for the validity of CGSI.

At the majority of excavation depths greater than 200 m, the CGSI models continue to show the best approximation of the explicit models (in terms of depth of yield). This observation is more consistent in the roof measurements than the walls, which is attributed to an in situ stress ratio of $K > 1$ and geometry effects of the arched adit with corners at the floor. The CGSI models deviate from the explicit models in shallow conditions at less than 200 m depth, which is explained by structurally controlled behaviour at low confinement that cannot be captured by continuum models. Indeed, no GSI approach is intended for use in this scenario.
Figure 3-30: Estimated depth of yield measurements (normalized to 3 m tunnel radius) from FEM models at a range of excavation depths show a better fit between the explicit and CGSI solutions when compared to the conventional worst case and joints only GSI approaches. The nonlinear data curves are best fits of the Carreau-Yasuda rheological model. The explicit rockmass structure and equivalent continuum GSI values in the FEM models are based on observations at the 600 m deep adit excavation face. The in situ stress conditions vary with depth (refer to Figure 3-27). The inset excavations show yielded material elements for the 100 and 2000 m deep worst-case equivalent continuum GSI models. The overbreak observations at excavation faces at 450, 550, and 600 m depths are best approximated using the CGSI equivalent continuum models.
3.9 Composite GSI Discussion

Two cases were used to support the Composite GSI (CGSI) method for complex rockmasses with multiple suites of structure as an improved method to determine rockmass strength from the conventional GSI approach. CGSI calculates a weighted harmonic average value from GSI assessments of individual suites of structure in a rockmass. Suites of rockmass structure are grouped by similar structure and surface or infill conditions.

A numerical based study used implicit models with GSI-based strength criteria of each suite of structure to calibrate stiffness and strength properties for the individual suites of explicit structures. The combined model with both calibrated suites of explicit structure was then compared to two implicit models of the whole rockmass represented by the conventional GSI and CGSI. In both calibration steps of elastic and plastic explicit models, the implicit CGSI model was an improved fit for the respective comparison criteria: elastic total displacement at the excavation boundary and plastic depth of yield from the excavation boundary.

While detailed numerical modelling where all rockmass structure is explicitly represented is the desired approach for geomechanical models, this approach remains outside of present computational capacities in cases with multiple suites of rockmass structure. A stepwise optimization procedure for the inclusion of multiple suites of rockmass structure in numerical models has been developed in conjunction with the Composite GSI approach. This optimization approach balances the explicit and implicit structure to provide a reasonable approximation of rockmass behaviour while working within computational limits. To apply this approach effectively, it is important to have a sound understanding of the rockmass and particularly its structure in order to accurately represent the rockmass behaviour and failure modes, as well as their impact on design. Two case examples of arched underground excavations were used to highlight the implications of modelling suites of rockmass structure explicitly or implicitly in FEM numerical models. In both cases, the fully explicit models revealed significant differences in rockmass behaviour compared to the partially and fully implicit models. The fully explicit model of a cavern in basaltic flows (Case A) showed considerably more joint failure propagation and remained within
computational limits. The rockmass behaviour shown in the fully explicit model of a tunnel in carbonate sedimentary strata (Case B) was captured by an intermediate model where the intrablock structure was modelled as an equivalent continuum material. The high computational demand of the explicit model, in this case, is not feasible for application at a larger scale project; therefore, the intermediate model coupled with a sound understanding of the rockmass failure mode would likely be sufficient for further analysis. This optimization technique with a subset of models can ultimately be implemented in the design of large-scale geotechnical projects, where smaller sample sections of a project can be tested through the full stepwise progression of models to ensure an appropriate understanding of the rockmass behaviour.

Finally, a validation case study of an adit at the El Teniente mine was used to compare the CGSI method to conventional GSI approaches. The FEM results of the CGSI models show better representations of both the explicit numerical models and site observations of excavation faces in the adit at 450, 550, and 600 m depths. Further models at various depths between 300 and 2000 m show the CGSI models provide more accurate estimates of yield depth than the conventional worst case and joints only GSI models when compared to the explicit model. The depth of yield for equivalent continuum models at depths less than 200 m deviate from the explicit models, which can be explained by structurally driven failure at low confinements that is not appropriate for any continuum approach. Overall, this case study provides evidence to validate the new CGSI method for rockmass characterization of complex rockmasses with multiple suites of structure as applied to behaviour evaluation using equivalent continuum FEM numerical models.
3.10 Accounting for Intrablock Structure using Intact Strength

In laboratory rock strength testing programs for geotechnical design, the Hoek-Brown failure envelopes can be defined from a suite of UCS, triaxial and/or tensile test data (Hoek and Brown, 1997). Although inconsistencies in data trends from a suite of tests may be a result of poor and/or variable testing practices, they may also be partly attributed, in complex rockmasses, to the presence and effect of intrablock structure on the strength of the intact laboratory sample. Standard sample selection practices target the most homogeneous and isotropic samples available, avoiding rockmass structures where possible. Where laboratory samples have been deemed to fail through structure instead of the matrix, they are typically discarded from the results. In rockmasses with ubiquitous intrablock structure, selection of a homogeneous sample that only contains matrix mineral grains is difficult to impossible. Instead of discarding test results for samples whose strength results are influenced by the presence of veins, this author proposes that such results be used to define complex rockmass strength more accurately.

Laboratory tests from the Esperanza mine in Chile are used in this section to analyze differences in strength properties between matrix and structure failure modes. Failure modes and strength profiles from tensile, UCS, and triaxial strength laboratory tests from Esperanza are sorted by failure mode. The implications of appropriate selection of data for use in design are illustrated using FEM simulations of a 10 m-diameter arched excavation at various depths.

3.10.1 Esperanza Laboratory Strength Tests

An example of tensile, UCS, and triaxial tests with corresponding best fit Hoek-Brown curves of a hydrothermally altered, primary (unleached and unoxidized) andesite from a copper porphyry deposit in northern Chile is shown in Figure 3-31. The data has been sorted by failure mode between intact rock (matrix) and veins (structure), and the respective Hoek-Brown criterion material properties are listed in Table 3-23. There is a clear difference in strength profiles between these failure modes. In this case, the variability about the mean structural failure envelope represents the effect of the various vein alterations present in the rock, including propylitic, potassic and quartz sericite.
Figure 3-31: Hoek-Brown strength envelopes for an altered andesite in a porphyry copper deposit, for failures through intact rock (solid lines) and hydrothermal vein intrablock structure (dashed lines); the inset photo shows an example of drill core with hydrothermal veins.
Table 3-23: Hoek-Brown properties of hydrothermal andesite sorted by laboratory test failure mode

<table>
<thead>
<tr>
<th>Generalized Hoek-Brown Parameter (Hoek et al., 2002)</th>
<th>Matrix Failure</th>
<th>Structural Failure (Veins)</th>
</tr>
</thead>
<tbody>
<tr>
<td>UCS (MPa)</td>
<td>90 (+/- 30)</td>
<td>20 (+/- 10)</td>
</tr>
<tr>
<td>m</td>
<td>18</td>
<td>12.5</td>
</tr>
<tr>
<td>s</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>a</td>
<td>0.5</td>
<td>0.5</td>
</tr>
</tbody>
</table>

Finite Element models of a 10 m diameter tunnel in this rockmass are used to illustrate the implications of using material properties from matrix or structural failure modes at depths of 100, 500, and 1000 m. In these models, the Generalized Hoek-Brown criterion was used to define the intact material strength based on each of the matrix and structure failure envelopes in Figure 3-31. The interblock structure joints in this example represent a rough, slightly altered surface condition. The model results, shown in Figure 3-32, indicate very similar yield behaviour between intact materials in shallow conditions (100 m depth). However, at 500 m and 1000 m depths, there is a significant difference in the extent of yield in both the material and explicit joint structure. At 1000 m, there is nearly twice the depth of plastic yield in the roof between the intact matrix material and the intact material properties with implicit intrablock structure. This finding has important implications for the effective selection of ground support.
Figure 3-32: FEM model results illustrate the implications of using intact material properties from matrix or structural laboratory testing failure modes on a 10 m diameter tunnel at a range of depths
3.11 Discussion and Concluding Remarks

Rockmass characterization is an essential component of geotechnical design. In complex rockmasses, this study has shown using several cases that it is important to include both interblock (e.g. joints and bedding) and intrablock (e.g. veins and stockwork) structures in the characterization process to more effectively capture and improve the prediction of rockmass behaviour. Observations of rockmass behaviour in underground excavations suggest that intrablock structure can dominate the overall behaviour at the excavation scale.

Application of a conventional characterization rationale to complex rockmasses would over-penalize the rockmass by using the worst case strength value; for example, by using the worst case in GSI of the structures and surface conditions present. While intrablock structure can dominate the behaviour in a rockmass, it may not weaken the rockmass to that extent. Development of the new Composite GSI (CGSI) approach to characterize complex rockmasses in this work has been numerically validated using simulations of an underground excavation and a field observation based case study. In both cases, the CGSI approach resulted in an improved estimate of rockmass behaviour in implicit equivalent continuum numerical models when compared to a conventional and conservative GSI approach.

The Composite GSI method was applied in conjunction with the development of a stepwise optimization procedure for numerical modelling of rockmasses with multiple suites of structure. A key aspect of the CGSI method that enables this stepwise optimization approach is the individual consideration of different suites of rockmass structures on the GSI chart before assembling a CGSI value for the rockmass as a whole. A balance of explicit structure and implicit structure (modelled as a continuum defined by CGSI properties) remains relevant for large scale geomechanical modelling that is controlled by the limitations of computational capacity. The optimal balance of the representation of structure maintains an accurate representation of rockmass behaviour and failure modes while remaining within computational limits.

An alternative method to account for intrablock structure is from the intact rock scale using intact rock strength properties from laboratory testing. The Hoek-Brown strength criterion (Hoek et al., 2002)
was selected because it directly incorporates rockmass observations through the lens of GSI and the parameters have direct relevance to rock behaviour and input parameters for numerical models. The intact rock scale approach modifies the intact strength parameters of the material based on UCS, triaxial, and tensile laboratory tests. This approach allows laboratory rock strength testing programs to account for intrablock structure by using results where strength properties are influenced by the presence of veins instead of discarding them as erroneous. The case study of laboratory test results sorted by failure mode through the intact matrix versus intrablock structure was applied to an excavation scale numerical simulation to illustrate the effects on depth of rockmass yield. At a shallow depth (100 m), the modelled rockmass behaviour was similar for both the matrix and intrablock failure modes. At greater depths (500 to 1000 m), however, there is a significant difference in the extent of yield in both the material and explicit joint structure. These findings support the observations that indicate intrablock structures in complex rockmasses have limited influence in shallow (low stress) conditions but dominate behaviour at depth (high stress).
3.12 References


Chapter 4
Brittle Overbreak Prediction for Hydrothermal Complex Rockmasses with Healed Structure

4.1 Introduction and Background
In sparsely jointed to massive hard rockmasses under high in situ and deviatoric stresses, brittle spalling is the dominant damage process and failure mode and can result in significant depths of overbreak outside the excavation design dimensions. The issue of spalling in mining and tunnelling excavations through homogenious rockmasses has been observed and examined by numerous authors, including Hoek (1968), Hoek and Brown (1980), Stacey (1981), Kaiser and McCreath (1993), Ortlepp (1997), Martin et al. (1997), Martin et al. (1999), Diederichs (2007), and others. As defined by Diederichs (2007), spalling is the development of visible extension fractures under compressive loading, which are parallel to the excavation boundary. In unsupported conditions and an anisotropic stress field (i.e. K ratio ≠ 1), maximum and minimum stresses develop around the excavation boundary. In cases where K > 1 and the horizontal stresses are greater than the vertical stresses, concentrations of high induced stress develop in the roof and floor of excavations (Figure 4-1). Here, the spalling process results in a notched geometry that nucleates at the maximum tangential stress locations around an excavation boundary.

In high stress conditions around a single excavation, brittle failure develops progressively as the excavation advances. This process is described by Martin et al. (1997) and illustrated here in Figure 4-2. Boundary parallel cracks form ahead of the tunnel face and begin to propagate in the tunnel, forming thin slabs initiated at a process zone. The thin slabs continue to form, developing a notched geometry, until confining stresses at the process zone are high enough to arrest crack development.

---

2 This chapter is in preparation for submission to an international journal and will have the following author list: Day, J. J., Diederichs, M. S., and Hutchinson, D. J.
Rock failure caused by brittle spalling can form significant overbreak around an excavation. It is, therefore, imperative to understand and predict this phenomenon for effective geotechnical design of excavations with respect to appropriate support and construction practices. Many researchers have addressed this issue for homogeneous rocks.

This study will demonstrate that the empirical predictive tools for homogeneous rocks are ineffective for heterogeneous complex rockmasses. Mechanistic-based predictive tools for different zones of failure for homogenous rockmasses are applied to explain the behaviour of heterogeneous rocks. Finally, new functions are developed to predict brittle overbreak in the analyzed heterogeneous complex rockmasses.

The complex rockmasses of focus in this study are in the New Mine Level Project at the El Teniente copper-molybdenum porphyry mine in Chile. The rockmass generally has sparse jointing but there are pervasive and ubiquitous intrablock structures in many lithological units. There are also brecciated units that present an alternative type of heterogeneity. Observations of overbreak in the undercut level show the presence of brittle failure, and detailed overbreak profile measurements are compared to existing predictive depth of failure tools and used to develop new predictive overbreak ranges for these complex rockmasses.
Figure 4-1: Boundary element elastic solution of induced stresses around a circular excavation, showing a highly stressed zone in the roof when the maximum principal stress, $\sigma_1$, is horizontal and perpendicular to the excavation axis.

Figure 4-2: Diagram of progressive formation of notch in roof of an excavation by brittle failure in highly deviatoric in situ stresses (Martin et al., 1997)
4.2 Predicting Brittle Spalling Overbreak

Brittle spalling predictions in homogenous rockmasses that are based on empirical data and mechanistic interpretations are discussed in this section.

4.2.1 Empirical Spalling Prediction

Several authors have compiled spalling observations into empirical prediction tools for engineering design applications. Hoek and Brown (1980) developed an empirical stability classification for square tunnels in South Africa with an in situ stress ratio of $K = 0.5$. This classification implements a ratio of major principal stress, $\sigma_1$, to laboratory unconfined compressive strength (UCS or $\sigma_c$). While useful for that locality, this tool is not readily transferable to other excavation shapes or stress conditions. Martin et al. (1999) converted this stress indicator into a general expression as the ratio between the maximum tangential boundary stress ($\sigma_{\text{max}}$) and $\sigma_c$: the damage index ($D_i$). The damage index suggests, based on the case histories used by Hoek and Brown (1980), the maximum rockmass strength near the excavation boundary is approximately $0.4\sigma_c$. This is in agreement with observations in massive granite by Read and Martin (1996) which suggest that maximum rockmass strength near the boundary is approximately $0.5\sigma_c$.

Martin et al. (1999) attributed the difference between intact laboratory and field rockmass strength to the differences in stress loading paths, where intact laboratory samples experience a simple monotonic load increase but excavation boundaries experience complex unloading from an in situ stress state and associated stress rotations.

A collection of case histories examined by Martin et al. (1999) were used to establish a relationship between the depth of brittle spalling failure and maximum tangential boundary stress. The case histories represent a range of stress conditions, rockmass conditions and lithologies, and tunnel geometries. The common elements of these cases are stress induced failure, moderately jointed to massive homogenous rockmasses, and brittle rockmass behaviour. The lithologies include andesites, quartzites, granites, mudstones, and limestones. The analysis resulted in a linear relationship between the dimensionless depth of failure and dimensionless maximum tangential stress, where the axes are
normalized to excavation radius and $\sigma_c$, respectively. This relationship also suggests that rockmass strength near the excavation boundary is approximately $0.4\sigma_c (\pm 0.1)$, which is in agreement with previous observations in different rockmasses.

Two important components of rock strength in addition to the unconfined compressive strength are damage (or crack) initiation (lower bound strength) and crack interaction (upper bound strength). According to Diederichs (2007), in situ strength corresponds to a threshold of extensile crack damage initiation; long-term yield strength corresponds to crack interaction. UCS laboratory test observations by Brace et al. (1966) suggest a range of crack initiation stress levels between 0.3-0.5 of UCS at various loading rates. Results of numerical modelling of crystalline Lac du Bonnet granite under simulated laboratory axial load by Diederichs (2007) support these observations with simulated acoustic emission observations. The range of crack initiation (CI) depends on the grain scale heterogeneity and nature of internal flaws. Crack interaction corresponds to the onset of nonlinearity in axial stress-strain laboratory measurements, which represents the interaction of previously independent cracks accumulating in the sample (Diederichs, 2003, 2007). Crack interaction has more recently become known as the Critical Damage (CD) threshold (Ghazvinian et al., 2012).

Diederichs (2007) added case histories and applied the established relationship between crack initiation, CI, and peak unconfined compressive strength, UCS or $\sigma_c$ (where CI $\approx 0.3-0.5\sigma_c$) to the empirical estimation tool for spalling depth around an excavation by Martin et al. (1999), as shown in Figure 4-3.
4.2.2 Mechanistic Interpretation of Spalling Prediction

Diederichs (2007) unravelled the mechanics of spalling to develop a mechanistic interpretation of the empirical spalling prediction criteria. The mechanisms responsible for reducing laboratory UCS strength to in situ strength investigated by Diederichs (2007) consist of pre-existing crack damage, stress rotation and damage, crack propagation and interaction, and internal stress heterogeneity and crack propagation. Diederichs (2007) used those investigation results to develop the composite damage initiation – spalling limit (DISL) brittle strength criterion (Figure 4-4). In this criterion, spalling occurs at low confinements and above the crack initiation (CI) strength envelope. The spalling limit boundary defines the transition to failure at high confinements above the critical crack damage (CD) (yield) strength envelope.
The difference in behaviour between laboratory samples and in situ conditions is also illustrated in Figure 4-4: crack propagation is suppressed in laboratory samples by a confining circumferential hoop tension while there is no propagation restriction near an excavation boundary.

Figure 4-4: Composite damage initiation - spalling limit (DISL) brittle constitutive model (solid line) synthesis of investigations into spalling mechanics; low confinement spalling limit is threshold between crack initiation strength limit in low confinement and crack damage / long term yield strength limit at high confinement; graphics illustrate crack behaviours in laboratory and in situ conditions (Diederichs, 2007)
Guidelines for the transition from the DISL approach for brittle failure (Diederichs, 2007) to the conventional Hoek-Brown and Geological Strength Index (GSI) approach for shear and squeezing failure (Hoek et al., 2002; Hoek and Marinos, 2000) were presented and tested by Diederichs (2007), and are reproduced here in Table 4-1. These guidelines are based on the rock strength ratio (compressive / tensile) for ranges of GSI.

Table 4-1: Guidelines for analyzing rock failure as shearing or spalling based on rock strength ratio (compressive / tensile) (Diederichs 2014, 2007)

<table>
<thead>
<tr>
<th>Strength Ratio</th>
<th>GSI &lt; 55</th>
<th>GSI = 55 – 65</th>
<th>GSI = 65 – 80</th>
<th>GSI &gt; 80</th>
</tr>
</thead>
<tbody>
<tr>
<td>UCS/T &lt; 9</td>
<td>Shear (GSI)</td>
<td>GSI</td>
<td>GSI</td>
<td>GSI</td>
</tr>
<tr>
<td>UCS/T = 9-15</td>
<td>GSI</td>
<td>GSI</td>
<td>GSI</td>
<td>GSI / DISL*</td>
</tr>
<tr>
<td>UCS/T = 15-20</td>
<td>GSI</td>
<td>GSI / DISL*</td>
<td>DISL / GSI*</td>
<td>DISL</td>
</tr>
<tr>
<td>UCS/T &gt; 20</td>
<td>GSI</td>
<td>GSI / DISL*</td>
<td>DISL</td>
<td>Spall (DISL)</td>
</tr>
</tbody>
</table>

*Ordering of methods indicates most appropriate

The DISL mechanistic approach for depth of failure prediction was compared by Diederichs (2007) to the linear fit to the empirical worst case depth of failure data (Figure 4-3) using inelastic numerical simulations. At crack initiation stresses of $\sigma_{\text{max}} / \text{UCS}$, the zero depth of failure matches the empirical correlation. With increasing tangential stress, the depth of failure progresses nonlinearly to a slope that is less than the empirical fit (Figure 4-5). This suggests that the linear fit overestimates the depth of failure in very high stress conditions. This trend is consistent with the unsupported and supported cases. The shear-based GSI strength approach, however, has positive nonzero depths of failure at crack initiation stresses, which does not agree with mechanical interpretations.
The GSI based failure predictions initiate at much lower stresses but ultimately underpredict depth of failure at higher stresses. The change from convex to concave slopes for the DISL and GSI depth predictions is explained by a fundamental difference between brittle and plasticity mechanics and strength criteria (Diederichs, 2007).

Perras and Diederichs (2015) investigated the suitability and sensitivity of the DISL approach to predict depths of excavation damage zones (EDZs) around circular excavations in brittle rock. EDZs are an essential consideration for the geomechanical design of deep geological repositories (DGRs) for the permanent storage of nuclear waste, where the effectiveness of the design is dependent on limiting fluid

Figure 4-5: Composite plot of mechanistic numerical simulations of spalling around an excavation using the DISL approach, compared to the empirical best fit mean and limits from Figure 4-3; depth of failure calculations using the conventional GSI rockmass (shear) approach for GSI illustrate fundamental difference between brittle and plasticity based strength criteria (Diederichs, 2007)
flow and therefore potential radionuclide transport through excavation parallel cracks. EDZs are divided into subsets based on damage and displacements, including the highly damaged zone (HDZ), inner EDZ (EDZ$_i$), and outer EDZ (EDZ$_o$), as shown in Figure 4-6.

![Figure 4-6: Excavation Damage Zone (EDZ) (modified after Ghazvinian, 2015)](image)

The highly damaged zone (HDZ) forms closest to the excavation boundary, has interconnected microfractures, and represents the inevitable damage as a result of geometry, structure, and/or induced stress changes (Perras and Diederichs, 2015). In brittle conditions, the HDZ cracks propagate instantaneously and rupture after initiation to form a notch geometry. The inner EDZ (EDZ$_i$) forms immediately outside the HDZ and is characterized by connected crack damage with significant dilation. The EDZ$_i$ contains significant damage and will form a notch geometry with aggressive scaling. The findings from Perras and Diederichs (2015) show the EDZ$_i$ limit correlates with the case histories based on maximum damage depth reported by Martin et al. (1999) and Diederichs (2007) in the empirical failure depth prediction chart, up to $\sigma_{max}/CI$ of approximately 1.75-2. The EDZ$_i$ gradually transitions to the outer EDZ (EDZ$_o$), which is characterized by partially connected to isolated damage (Bossart et al., 2015).
2002) and without significant dilation (Perras and Diederichs, 2015). The EDZ, is the limit of damage initiation which is usually minor in homogeneous rocks.

The farthest afield excavation influence zone (EIZ) involves only elastic change (Siren et al., 2015). It is therefore of little significance to brittle yield for a single excavation and, for this reason, was not included in the brittle damage zone depth analysis by Perras and Diederichs (2015).

The numerical investigation based on laboratory testing and statistical analyses by Perras and Diederichs (2015) resulted in predictive equations for depths of failure for EDZ zones relevant to single excavations in brittle rock. The mean equations with error values are:

\[
EDZ_0 / R = 1 + 0.6(\pm 0.07)(\sigma_{\text{max}} / CI - 1)^{0.6(\pm 0.04)}
\]  
(4.1)

\[
EDZ_i / R = 1 + 0.4(\pm 0.07)(\sigma_{\text{max}} / CI - 1)^{0.5(\pm 0.07)}
\]  
(4.2)

\[
HDZ / R = 1 + 0.2(\pm 0.06)(\sigma_{\text{max}} / CI - 1)^{0.7(\pm 0.25)}
\]  
(4.3)

These equations were developed following sound mechanistic interpretations using investigations of relatively homogeneous rocks: granite, limestone, and mudstone. The upper 68% interval of these equations represents the maximum potential depth of failure and is in good agreement with the in situ EDZ case studies reported by Perras and Diederichs (2015), as shown in Figure 4-7. The theoretical DISL approach by Diederichs (2007) and brittle EDZ investigation by Perras and Diederichs (2015) form fracture mechanics based limits for depth of brittle failure for homogeneous rocks.
Figure 4-7: The empirical depth of failure linear fit from Diederichs (2007) superimposed by the upper 68% prediction intervals of numerically based EDZs that represent the maximum EDZ depth, compared to in situ case histories of EDZ measurements (Perras and Diederichs, 2015)
4.3 Drift Overbreak Observations at El Teniente Mine

The New Mine Level Project at the El Teniente Cu-Mo porphyry block cave mine is, on average, over 1000 m deep and the rockmass exhibits stress-driven brittle behaviour. At the time of this investigation, the excavation infrastructure is under construction and ore production has not yet commenced. The excavation requirements for El Teniente and other large block cave mines require hundreds of kilometers of drifts, ramps, and raises between levels. The infrastructure footprints of all caving levels at El Teniente are shown in Figure 4-8. In the New Mine Level Project, multiple levels are required under the footprint of the cave zone to house ore extraction, transportation, and other infrastructure. These include the undercut level, production level, haulage level, and ventilation level. The undercut level is immediately below the targeted orebody where upward vertical drill holes will host the explosives that initiate caving and production (Figure 4-9). This level is the target area of this investigation. All of this infrastructure must be excavated before mining production begins.

![Figure 4-8: 3-Dimensional model of the El Teniente mine, showing the undercut levels (modified after Pardo et al., 2012)](image-url)
A visit to the mine on October 16th, 2014 provided an excellent opportunity to observe the behaviour of the rockmass at different locations in the undercut level of the New Mine Level Project. A network of drifts in this level cover the footprint of the target orebody, and additional drifts for operational infrastructure and transportation corridors must also be constructed. The complex geometry of the orebody and infrastructure planning results in excavations that are in a variety of orientations.

In this deep environment with highly deviatoric stresses, overbreak above the planned drift dimensions is dominated by stress-driven brittle failure processes. Jarufe and Vasquez (2014) reported in situ principal stress orientations that are in agreement with measurements by Windsor et al. (2006), and Diederichs (2016) analyzed in situ stress magnitudes appropriate for the New Mine Level, as listed in Table 4-2. The excavation orientations for the major undercut drifts in the New Mine Level are planned to be predominantly parallel to the maximum principal stress, $\sigma_1$, to minimize induced deviatoric stresses around the primary excavations.
Table 4-2: In situ stresses for studied section of New Mine Level undercut level

<table>
<thead>
<tr>
<th>Principal Stress</th>
<th>Magnitudes (MPa) (after Diederichs, 2016)</th>
<th>Trend (after Jarufe and Vasquez, 2014)</th>
<th>Plunge (after Jarufe and Vasquez, 2014)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_v = \sigma_3$</td>
<td>32.4</td>
<td>155</td>
<td>90</td>
</tr>
<tr>
<td>$\sigma_h = \sigma_2$</td>
<td>41.6</td>
<td>245</td>
<td>0</td>
</tr>
<tr>
<td>$\sigma_H = \sigma_1$</td>
<td>60.0</td>
<td>335</td>
<td>0</td>
</tr>
</tbody>
</table>

The in situ stress K ratio of ~2 will result in overbreak that is dominantly in the roof of the excavations and lower stress zones in the walls. Therefore, overbreak in the roof and up to 45° on either side of vertical is the focus of this investigation. An overview of overbreak observations around excavations in different orientations is shown in Figure 4-10. Larger overbreak was observed as notch geometries from brittle spalling failure (as in locations 4a, 4b, and 5b), which occurred in excavation orientations sub-perpendicular and perpendicular to the major principal stress, $\sigma_1$. Smaller depths of overbreak were observed approximately parallel to the maximum principal stress (e.g. locations 2b, 3b, and 5a). Minor boundary-parallel spalling was observed in the roof near the face in these locations (3b is highlighted in Figure 4-13a). This process follows investigations of brittle crack propagation investigated by Diederichs (1999, 2007) that is highly sensitive to low confinement conditions (Figure 4-13b). Detailed overbreak observations at each Station are discussed in the following subsections.
Figure 4-10: Overview of overbreak observations around excavations of different orientations in the New Mine Level undercut level at the El Teniente copper porphyry mine on October 16th, 2014. Larger overbreak was observed as notch geometries in directions sub-perpendicular to the maximum principal stress (e.g. Stations 4a, 4b, and 5b), while smaller overbreak was generally observed parallel to the maximum principal stress (e.g. Stations 2b, 3b, and 5a)
4.3.1 Station 2
Observations at Station 2 are shown in Figure 4-11. The initiation point of notch formation is visible in 2a immediately behind the drift heading. The overbreak increases significantly at the drift intersection just a few meters behind the 2a face. Upon further advance of drift 2a, the notch formation is expected to propagate. In contrast, the overbreak shapes along the 2b drift axis are rounded without notch formation due to the reduced deviatoric stresses at the excavation boundary.

Figure 4-11: Overbreak observations at Station 2 in the New Mine Level undercut level at El Teniente on October 16th, 2014. Drift heading 2a parallel to $\sigma_2$ has notch overbreak, while 2b drift parallel to $\sigma_1$ has a rounded overbreak profile
4.3.2 Station 3

Observations at Station 3 are shown in Figure 4-12. Drift heading 3 is parallel to $\sigma_1$ and has a rounded overbreak profile. The unsupported roof immediately behind the face shows partially formed brittle spall slabs that are parallel to the excavation boundary. This behaviour is classified between stages 2 and 3 of notch formation by Martin et al. (1997): between the process zone, and spalling and slabbing. The lower deviatoric stresses in this heading may prevent significant notch formation as the drift advances.

![Figure 4-12](image)

Figure 4-12: Overbreak observations at Station 3 in the New Mine Level undercut level at El Teniente on October 16th, 2014. Drift heading is parallel to $\sigma_1$ and has rounded overbreak profile. The detail view of the unsupported roof at the heading shows partially formed brittle spall features parallel to the excavation boundary
4.3.3 Station 4

Observations at Station 4 are shown in Figure 4-14. Station 4 is located along a U-shaped ramp and is therefore exposed to transitioning orientations from being parallel to \( \sigma_1 \) to parallel to \( \sigma_2 \). The overbreak changes between rounded and notched profiles, respectively, between these orientations. Fully formed notches occur in 4a, 4e, and the foreground of 4d. The heading in the background of 4d does not have a notch; the notch is expected to continue development as the heading advances, as per the longitudinal advance behaviour described in Figure 4-2 by Martin et al. (1999). Rounded profiles as in 4b, 4c, and 4f occur when the drift is parallel to \( \sigma_1 \), which is consistent with other observations at the other stations.
Figure 4-14: Overbreak observations at Station 4 in the New Mine Level undercut level at El Teniente on October 16th, 2014. The shape and extent of overbreak profiles changes at different orientations along the ramp with respect to principal stress orientations. Irregular notch profiles form when the drift is parallel to $\sigma_2$ (as in 4a, 4d, and 4e) while rounded profiles occur when the drift is parallel to $\sigma_1$ (as in 4b, 4c, and 4f)
4.3.4 Station 5

Observations at Station 5 are shown in Figure 4-15. The 5a drift is oriented subparallel to $\sigma_2$ and exhibits irregular overbreak profiles in both magnitude and orientation along the chainage. The irregular overbreak is attributed to the heterogeneity in the surrounding stockwork mafic complex unit caused by the hydrothermal quartz vein stockwork intrablock structure. Like the heading in Station 4d, the overbreak does not form above heading 5c immediately at the drift heading. This process zone, where shearing and crushing of the rock occurs (Martin et al., 1997 and Figure 4-2), nucleates the notch formation as the heading advances, which is visible just a few meters behind the face. Drift heading 5b is parallel to $\sigma_1$ which, like the other drifts in this orientation, has not formed a notch overbreak geometry.

Figure 4-15: Overbreak observations at Station 5 in the New Mine Level undercut level at El Teniente on October 16th, 2014
4.4 Overbreak Profile Measurements

Drift profile measurements along two drifts in the undercut level of the New Mine Level at El Teniente are used in this study to examine overbreak. The two drifts, A and B, are oriented sub-perpendicular to each other and are located in four different lithologies. Plan views of drifts A and B are shown in Figure 4-16, including the lithologies and orientations with respect to in situ principal stresses.

Drift A is horizontal, trending at 335° and parallel to the maximum principal stress, $\sigma_1$, and passes through three lithology units: the stockwork mafic complex, dacite porphyry, and anhydrite breccia. There is also a section of the drift that cuts through a contact between the stockwork mafic complex and dacite porphyry. Overbreak measurements were recorded approximately every 5 m and advancing toward 335° along the 56 m segment of chainage by the mine surveyors. A summary of these overbreak profiles,
measured at 15° intervals around each profile, is shown in Figure 4-17. Here, the entire profile with all units is shown in addition to overbreak profiles in each lithology. A mean and range analysis of these overbreak profiles by lithology is shown in Figure 4-18. The maximum overbreak measurements vary significantly in magnitude and orientation by lithology, and exhibit notch geometries in the stockwork mafic complex and dacite porphyry units. The mean and minimum overbreak measurements are more consistent, however, with only a slight elongation toward the notch geometries in the stockwork mafic complex and dacite porphyry units.

Drift B is horizontal and trending eastward at 090° and sub-parallel to the intermediate principal stress, σ₂. It only passes through the stockwork mafic complex unit. Overbreak measurements were recorded approximately every 5 m and advancing eastward along a 340 m segment of chainage by the mine surveyors. A summary of these overbreak profiles, measured at 15° intervals around each profile, is shown in Figure 4-19. A mean and range analysis of these overbreak profiles is shown in Figure 4-20. The maximum overbreak measurements show a very large notch peaking at 15° north of vertical around the drift profile. The mean overbreak measurements exhibit a more pronounced elongation in the direction of the notch geometry than the mean measurements in Drift A due to the higher deviatoric stresses near the excavation boundary.

The maximum overbreak profile measurements in the stockwork mafic complex measure 2 m in drift A (parallel to σ₁) and reach 3 m in drift B (sub-parallel to σ₂). This difference is consistent with the author’s observations of relative overbreak magnitude in different drift orientations at Stations 2 to 5 in the New Mine Level.
Figure 4-17: Overbreak profiles along the undercut level drift A (parallel to $\sigma_1$) in various lithologies
Figure 4-18: Range of overbreak along drift A through (a) all units, and (b-e) the individual units.
Figure 4-19: Overbreak profiles along undercut level drift B (subparallel to $\sigma_2$) in stockwork mafic complex geological unit in the New Mine Level at El Teniente

Figure 4-20: Range of overbreak along drift B entirely through the stockwork mafic complex unit
4.4.1 Geological Descriptions of Units

The lithological units encountered in this study include the stockwork mafic complex, dacite porphyry, and anhydrite breccia. The lithological and petrographical descriptions of these units and their hydrothermal alterations were documented by the El Teniente Division of Codelco during their exploration of the New Mine Level project zone and are summarized in the following sections.

4.4.1.1 Stockwork Mafic Complex

The stockwork mafic complex unit is comprised of gabbro, diabase, and basaltic porphyry intrusives (e.g. Skewes et al., 2002). The age has been dated to the late Miocene Epoch (8.3 to 5.59 Ma) (Munizaga et al., 2002). The difference between the gabbro, diabase, and basaltic components of the unit is primarily textural. The diabase (Figure 4-21a) has a porphyritic texture composed of subhedral plagioclase crystals with 2-5 mm diameter and a fine-grained matrix of < 1 mm diameter. The gabbro (Figure 4-21b) consists of fine-grained phaneritic, equigranular plagioclase of ≤ 1 mm diameter. These crystal sizes are similar to the fine-grained matrix in the diabase. The basalt (Figure 4-21c) is coarser grained with 3-4 mm long phenocrysts of plagioclase and a fine-grained aphanitic mass that ranges from 35-70% of the rock composition. The plagioclase has been partially replaced by sericite.

Hydrothermal alteration is similar in all three intrusives where penetrative biotite mainly affects the fine grained plagioclase, resulting in a black to dark brown colour. Quartz and magnetite alterations are cloudy light grey in colour. Other alteration minerals include anhydrite and chlorite, ± sericite, tourmaline and/or actinolite. Opaque minerals include magnetite and pyrite ±chalcopyrite and/or bornite.

At the time of borehole exploration of the New Mine Level, the gabbro phase of the stockwork mafic complex unit was expected to dominate in the New Mine Level. The contact relationships between the gabbro, diabase, and basalt phases are not clear at the mine scale but can be distinguished in drill core by sharp texture changes and minor brecciation.
4.4.1.2 Dacite Porphyry
The dacite porphyry unit (Figure 4-21d) is a felsic intrusive that is light grey to white in colour and has a porphyritic texture. Phenocrysts are composed of oligoclase, euhedral books of biotite, and subhedral quartz grains (Cuadra, 1986). The age has been dated to the early Pliocene Epoch: 4.6-4.7 Ma using the K/Ar method (Clark et al., 1983; Cuadra, 1986), and 5.28 ± 0.1 Ma using the U/Pb method (Munizaga et al., 2002).

This unit has formed in an N to NNW trending stock with a subvertical dip, located on the north side of the central Braden Breccia Pipe. It intruded the stockwork mafic complex and was intruded later by the Braden Breccia, which is a chimney of subvolcanic breccias that post-dates mineralization.

The contacts with the stockwork mafic complex are either sharp or brecciated. When brecciated, fragments of subangular to subrounded mafic rock are in sharp contact with a dacite porphyry matrix.

4.4.1.3 Anhydrite Breccia
The anhydrite breccia unit is part of the breccia complex and preferentially forms from porphyries intruding the stockwork mafic complex, incorporating fragments of mafic and felsic rocks (Figure 4-21e). Anhydrite and quartz are the major and minor constituents, respectively, of the cement matrix, which are from hydrothermal fluids. This unit formed during the early stages of the deposit formation.

This unit can be monomictic with clasts from the stockwork mafic complex, dacite porphyry, or diorite porphyry. It can also be polymictic with clasts from the stockwork mafic complex and the dacite porphyry, or clasts from the stockwork mafic complex and the diorite porphyry. The clast shapes range from subangular to subrounded.

Notably, this unit has a vertical zonation where the amount of quartz in the cement matrix increases to be the major constituent with increasing depth. This has been observed at depths greater than Level Tte. 8, which is approximately 100 m above the New Mine Level (Figure 4-8). This unit maintains a consistent name with depth for the purposes of spatial modelling at the mine.
Figure 4-21: Examples of lithologies studied at El Teniente: Stockwork mafic complex (a) diabase, (b) gabbro, and (c) basalt porphyries; (d) dacite porphyry; (e) anhydrite breccia with clasts from the stockwork mafic complex unit; and (f) diorite porphyry (photographs courtesy of Codelco)

4.4.1.4 Diorite Porphyry

The diorite porphyry unit is white to greenish white in colour with a porphyritic texture. The phenocrysts are composed of plagioclase and biotite. It occurs as a series of smaller stocks and dykes trending 330° with a large vertical extent. The age has been dated to 7.4 – 7.1 Ma using the K/Ar method (Cuadra, 1986) and 5.7 Ma using the Ar40/Ar39 method (Maksaev et al., 2001). In the New Mine Level, the diorite porphyry is typically greying white to brownish grey in colour (Figure 4-21f). Along with the plagioclase and biotite phenocrysts that are present in the rest of the mine, quartz crystals occasionally occur. The greyish white rocks are mainly composed of euhedral to subhedral plagioclase (30%) that has been altered by sericite. Anhedral biotite phenocrysts are also present. The fine-grained mass is composed of 40-50%
plagioclase, 15-25% quartz, and 3% biotite that has some chlorite alteration. Minor components of anhydrite, epidote, and opaque minerals are also present.

Like the dacite porphyry, the diorite porphyry has sharp contacts with the stockwork mafic complex. Igneous breccias can occur in the contact zones with a matrix of dacite porphyry and clasts of the stockwork mafic complex.

The diorite porphyry itself was not encountered as a mapped unit in this study. However, its description is included here for completeness since it can occur as a clast constituent of the anhydrite breccia.

**4.4.2 Hydrothermal Alteration of Units**

The primary forms of mineralization and hydrothermal alteration at El Teniente are networks of veins (stockwork) that provided pathways for secondary mineral deposition and alteration of primary minerals. The stockwork originated with the emplacement of intrusive igneous bodies and the Braden breccia pipe and additional vein types formed during subsequent hypogene hydrothermal alteration. Stockwork in the New Mine Level project is dominated by first generation late magmatic vein types with largely quartz mineralization, and minor occurrences of chalcopyrite and magnetite veins (Figure 4-22). Some veins have halos composed of quartz. Less altered rocks typically have sparse occurrences of veins.

![Figure 4-22: Examples of early alteration phase vein types in the New Mine Level at El Teniente; (a) quartz ± anhydrite, biotite, chalcopyrite, molybdenum, bornite, pyrite, chlorite (without halo), (b) quartz-biotite-anhydrite-chlorite ± chalcopyrite, pyrite, biotite (silica or silica-chlorite halo with adjacent disseminated biotite); (c) chalcopyrite and/or bornite and/or pyrite and/or molybdenite ± anhydrite, quartz, biotite (without halo); (d) magnetite ± quartz, biotite (silica halo) (courtesy of Codelco)](image-url)
The stockwork mafic complex is commonly altered by biotite that occurs as part of vein halos or disseminated in the matrix. Quartz and anhydrite also replace sections of the matrix. The primary plagioclase may be partially or entirely replaced by sericite. This unit contains an especially dense and ubiquitous network of quartz veins with varying thicknesses (< 1 mm to 50 cm) and orientations. The larger scale quartz veins have a very planar geometry that is often longer than a drift span and spaced on the meter scale, and are considered to have formed during the cooler temperature (late) stages of hydrothermal alteration. Smaller scale quartz veins in the same network form more complex polygons on the decimeter scale, and are considered to have formed during the hotter temperature (early) stages of hydrothermal alteration. The different generations of quartz vein formation correlates to different failure behaviour, where the cool, large-scale planar quartz veins have relatively weak wall rock bonds when compared to the hot, small-scale tortuous quartz veins.

The dacite porphyry is altered by a variety of minerals including silica, biotite, chlorite, and anhydrite. Chlorite is an alteration product of the primary biotite. Petrographic analysis reveals sericite and calcite alteration of plagioclase. Sericite and clay are present in fracture zones and cleavage. Close to contacts with the stockwork mafic complex, the disseminated biotite is present in both units. The diorite porphyry is primarily altered by sericite and chlorite, with minor anhydrite and polycrystalline quartz. The dacite porphyry contains less and thinner veins than the stockwork mafic complex.

In the case of the anhydrite breccia, the hydrothermal alteration is part of the main unit description since the anhydrite and quartz alteration products form the matrix around the igneous porphyry clasts. Additional veins are relatively sparse.

The orebody zone above the New Mine Level undercut level is a continuation of the mineralization recognized at shallower mine levels, including primarily sulphide chalcopyrite and bornite with some pyrite. These ore minerals are both hosted in veins and disseminated in the matrix.
4.5 Application of Spalling Prediction to the Complex Heterogeneous Rockmasses

The early empirical estimations of the depth of brittle failure around an excavation (e.g. Hoek and Brown, 1980, Martin et al., 1999) are based on cases of homogeneous massive to moderately jointed rockmasses. The mechanistic interpretations of these predictive results by Diederichs (2007) and Perras and Diederichs (2015) are also based on homogeneous massive rocks such as granite, limestone, and mudstone. The heterogeneities found in complex rockmasses with intrablock structure reduce the validity of these estimation tools for depth of failure. The complex hydrothermal intrablock structures at the El Teniente porphyry mine result in distinctly heterogeneous rockmasses. The consequence of this heterogeneity is evident in the erratic nature of the maximum overbreak profiles from drifts A and B (in this study) in the New Mine Level undercut level (Figure 4-18 and Figure 4-20).

The heterogeneity is also visible in laboratory samples, as shown in Figure 4-23. Failure of laboratory samples in compression often forms along veins as a result of stiffness contrast between the matrix and vein infill minerals, which is evident in these tests. UCS laboratory test data is combined with maximum overbreak depths to compare the units examined at El Teniente to the empirical linear prediction for depth of failure by Diederichs (2007) and Martin et al. (1999). Each unit examined has four data points of UCS values from two suites of laboratory test data. One laboratory test program had approximately ten samples for each unit; therefore, the minimum, mean, and maximum values were applied to this analysis. Example photographs of samples tested in this program are shown in Figure 4-23. The other suite of laboratory test data only reported mean values in the database, which is the fourth data point for each unit in this analysis. The UCS values selected for this analysis are listed in Table 4-3. The in situ stress magnitudes used to calculate $\sigma_{\text{max}}$ for case drifts A and B are listed in Table 4-4.

The maximum depth of failure cases for each lithology in drifts A and B correlate poorly to the empirical linear fit of depth of failure by Diederichs (2007, 2010), as shown in Figure 4-24. Although these drifts are part of a complex excavation network plan, at the time of this investigation, the drifts under consideration were excavated very early in the development of the level and can, therefore, be treated as single excavations.
Figure 4-23: Intact and failed laboratory UCS sample pairs of three lithologies at El Teniente (Courtesy of M. S. Diederichs)

Table 4-3: UCS values from laboratory test data for each lithology used in range for predicted depths of spalling

<table>
<thead>
<tr>
<th>UCS measurement (MPa)</th>
<th>Stockwork mafic complex</th>
<th>Dacite porphyry</th>
<th>Stockwork mafic complex – dacite porphyry contact</th>
<th>Anhydrite breccia</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum</td>
<td>99</td>
<td>76</td>
<td>76**</td>
<td>68</td>
</tr>
<tr>
<td>Mean</td>
<td>108</td>
<td>110</td>
<td>109*</td>
<td>91</td>
</tr>
<tr>
<td>Maximum</td>
<td>113</td>
<td>157</td>
<td>157**</td>
<td>112</td>
</tr>
<tr>
<td>Database</td>
<td>120</td>
<td>110</td>
<td>115*</td>
<td>102</td>
</tr>
</tbody>
</table>

* Mean of values for each lithology
** Widest range of combined minima and maxima for each lithology

Table 4-4: In situ stress magnitudes and orientations for case drifts A and B in the El Teniente New Mine Level

<table>
<thead>
<tr>
<th>Principal stresses (MPa)</th>
<th>Drift A</th>
<th>Drift B</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_1$</td>
<td>60 (horizontal, parallel to drift)</td>
<td>56.8 (horizontal, perpendicular to drift)</td>
</tr>
<tr>
<td>$\sigma_2$</td>
<td>41.6 (horizontal, perpendicular to drift)</td>
<td>45.2 (horizontal, parallel to drift)</td>
</tr>
<tr>
<td>$\sigma_3$</td>
<td>32 (vertical)</td>
<td>32 (vertical)</td>
</tr>
<tr>
<td>$\sigma_{\text{max}}$</td>
<td>92.4 (tangential to excavation boundary)</td>
<td>138.4 (tangential to excavation boundary)</td>
</tr>
</tbody>
</table>

The stockwork mafic complex unit is observed in both drifts A and B (sub-perpendicular orientations) and in both cases the maximum overbreak is entirely greater than the empirical trend and its error bars. The dacite porphyry is observed in drift A and the range of overbreak straddles the empirical trend. The anhydrite breccia is observed in drift A and the overbreak is entirely less than the empirical
trend and its error bars. The zone of contact between the stockwork mafic complex and dacite porphyry observed in drift A presents overbreak similar to that of the anhydrite breccia.

Figure 4-24: Empirical linear fit for brittle failure prediction of homogeneous massive to moderately jointed rockmasses after Diederichs (2007, 2010) compared to new maximum depth of failure cases observed at the El Teniente undercut level (drifts A and B) in four lithologies and two orientations; the poor correlation is attributed to the heterogeneous character of the El Teniente rockmasses.

Diederichs (2007) and Perras and Diederichs (2015) showed that a mechanistic-based approach for depth of failure is nonlinear with respect to stress and has a poor correlation to the empirical linear trend (Figure 4-5 and Figure 4-7). To explain the overbreak measurements at El Teniente using a mechanistic rationale, the nonlinear EDZ trends developed by Perras and Diederichs (2015) are compared to the El Teniente data. In addition to the maximum overbreak cases from El Teniente that were compared
to the empirical trend, means and standard deviations for overbreak measurements and stress conditions have been calculated for each lithology and drift. The standard deviation ranges of stress conditions are based on the four laboratory UCS samples also used in the maximum overbreak analysis (see Table 4-5). The means and standard deviations of the overbreak measurements are calculated using the maximum depth of failure measurements at seven angles around the drift profile. These angles are spaced at 15° intervals between 45° and 135° around the excavation where 90° is vertical upward (as in Figure 4-17 to Figure 4-20). These depths of overbreak calculations, normalized to a drift radius of 2.2 m, are shown in Table 4-6 for each lithology.

Table 4-5: Calculated means and ranges of maximum tunnel stress / UCS for each lithology

<table>
<thead>
<tr>
<th>(\sigma_{\text{max}} ) / UCS</th>
<th>Drift A: Stockwork mafic complex</th>
<th>Drift A: Dacite porphyry</th>
<th>Drift A: Stockwork mafic complex – dacite porphyry contact</th>
<th>Drift A: Anhydrite breccia</th>
<th>Drift B: Stockwork mafic complex</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean</td>
<td>0.84</td>
<td>0.87</td>
<td>0.86</td>
<td>1.03</td>
<td>1.26</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>0.07</td>
<td>0.26</td>
<td>0.26</td>
<td>0.24</td>
<td>0.10</td>
</tr>
</tbody>
</table>

Table 4-6: Normalized depths of overbreak (r/a) for each lithology with drift radii (a) of 2.2 m

<table>
<thead>
<tr>
<th>Normalized depth of overbreak (r/a) by angle around drift</th>
<th>Drift A: Stockwork mafic complex</th>
<th>Drift A: Dacite porphyry</th>
<th>Drift A: Stockwork mafic complex – dacite porphyry contact</th>
<th>Drift A: Anhydrite breccia</th>
<th>Drift B: Stockwork mafic complex</th>
</tr>
</thead>
<tbody>
<tr>
<td>45</td>
<td>1.50</td>
<td>1.73</td>
<td>1.18</td>
<td>1.32</td>
<td>1.64</td>
</tr>
<tr>
<td>60</td>
<td>1.59</td>
<td>1.45</td>
<td>1.11</td>
<td>1.34</td>
<td>1.70</td>
</tr>
<tr>
<td>75</td>
<td>1.86</td>
<td>1.70</td>
<td>1.16</td>
<td>1.32</td>
<td>1.91</td>
</tr>
<tr>
<td>90 (vertical)</td>
<td>1.68</td>
<td>1.73</td>
<td>1.18</td>
<td>1.36</td>
<td>2.18</td>
</tr>
<tr>
<td>105</td>
<td>1.55</td>
<td>1.45</td>
<td>1.23</td>
<td>1.34</td>
<td>2.36</td>
</tr>
<tr>
<td>120</td>
<td>1.61</td>
<td>1.34</td>
<td>1.27</td>
<td>1.36</td>
<td>2.20</td>
</tr>
<tr>
<td>135</td>
<td>2.05</td>
<td>1.41</td>
<td>1.23</td>
<td>1.34</td>
<td>1.68</td>
</tr>
<tr>
<td>Mean</td>
<td>1.69</td>
<td>1.55</td>
<td>1.19</td>
<td>1.34</td>
<td>1.95</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>0.20</td>
<td>0.17</td>
<td>0.05</td>
<td>0.02</td>
<td>0.29</td>
</tr>
</tbody>
</table>

The El Teniente data is compared to the upper 68% depth of failure prediction intervals of numerically based EDZs by Perras and Diederichs (2015) that represent the maximum EDZ depth when compared to in situ case measurements, as shown in Figure 4-25.
Figure 4-25: Ranges of overbreak depths for case drifts A and B at the El Teniente New Mine Level Project. Means and standard deviations for each lithology are plotted along with the absolute maximum overbreak measurements (see Figure 4-18 and Figure 4-20). Data is compared to the upper 68% depth of failure prediction intervals of numerically based EDZs by Perras and Diederichs (2015) that represent the maximum EDZ depth when compared to in situ measurements, as well as the empirical linear fit by Diederichs (2010) and after Martin et al. (1999). The overbreak measurements from El Teniente vary by lithology with respect to the existing prediction functions due to the complex intrablock structures present.

In all four lithologies, the upper 68% of the EDZ predictive functions by Perras and Diederichs (2015) result in an improved correlation to the means and standard deviations of overbreak depth. The difference in overbreak for the maximum failure cases, however, varies between units, which is driven by the variation in intact rock and intrablock structure heterogeneity. The stockwork mafic complex exhibits
the largest depth of overbreak and has the best correlation to the EDZₐ, 68% limit. The mean failure depth in the dacite porphyry is in correlates to both the empirical linear fit and the DISL-based EDZ, limit. The range of data for the dacite porphyry is a better fit to the EDZᵢ limit than the empirical linear trend. The anhydrite breccia and zone of contact between the stockwork mafic complex and dacite porphyry are a best fit to the HDZ limit. The differences in geology and alteration features in these units explain their correlations to different EDZ limits, which is discussed in detail presently.

The stockwork mafic complex unit contains hydrothermal veins that are dominated by quartz mineralogy. The large-scale, late stage quartz vein intrablock structure is characterized by a smooth, planar geometry with poor adhesion to the wall rock, relative to early stage small-scale quartz veins. The contrast between the intact mafic matrix and quartz veins in this unit, in addition to the planar and persistent vein geometry, allow for maximum crack propagation after initiation and immediate strength loss. When present, the alteration mineralogy near the veins in this unit typically occurs as biotite, which is a relatively weak and platy mineral that would aid in rapid crack redirection and propagation near and along matrix-vein contacts. Furthermore, the contrasting stiffness between wall rock and veins would redirect initiating cracks to optimal orientations along the planar contact boundaries for maximum propagation. Therefore, full overbreak tends to occur at the first sign of damage. In this case, the EDZₐ limit for the depth of failure identified by Perras and Diederichs (2015) is most suitable to represent the stockwork mafic complex unit. In homogeneous rocks, the EDZₐ is the limit of damage initiation that is normally minor and without significant dilation. In the stockwork mafic complex, however, damage initiation triggers rupture along the intrablock structure, and the planar geometry of the veins encourages direct and planar crack propagation.

The dacite porphyry unit has fewer stockwork veins than the stockwork mafic complex because it intruded after the early stage hydrothermal veins had already formed in the stockwork mafic complex and other older units. In this case, the EDZᵢ limit for the depth of failure by Perras and Diederichs (2015) is the most suitable. The EDZᵢ limit represents significant damage to the rockmass where a brittle spall
notch geometry will form with encouragement from aggressive scaling during excavation. Most unsupported damage in homogenous rocks follows the EDZ trend.

The anhydrite breccia unit contains large clasts of contrasting mineralogy in the intact anhydrite and quartz matrix. This unit exhibits minimal overbreak since crack propagation is largely arrested by the large aggregate material. This phenomenon is similar to the strengthening effect of large aggregate clasts in concrete when compared to homogeneous and fine grained cement, where there is no agent present to arrest crack propagation. The HDZ limit represents instantaneous damage and guaranteed formation of a notch geometry (Perras and Diederichs, 2015). Overbreak in the anhydrite breccia unit is limited to instantaneous damage because of the suppression of crack propagation.

The contact zone between the stockwork mafic complex and dacite porphyry shows similar overbreak patterns when compared to the anhydrite breccia. Although the individual stockwork mafic complex and dacite porphyry units experience significantly larger amounts of overbreak due their geology that allows for or even encourages crack propagation, the contact zone in the measured drift was likely brecciated by the intrusion of the dacite porphyry. Examples of this contact zone have been observed in different areas of the mine (see Figure 4-26). In this case, the majority of veins would be limited to the clasts of the stockwork mafic complex, which nullifies the promotion of crack propagation through their long and planar parent geometry. Additionally, the stiffness contrast between the dacite porphyry matrix and the stockwork mafic complex clasts would arrest crack propagation, similar to the anhydrite breccia. Finally, the constituents of this mechanically brecciated contact zone have a higher strength than the anhydrite breccia, which further reduces the overbreak depth of failure. Therefore, like the anhydrite breccia, this contact zone unit is also best captured by the HDZ limit defined by Perras and Diederichs (2015).
Figure 4-26: Mechanically brecciated contact zone where the dacite porphyry intruded the stockwork mafic complex that was previously altered and veined, in (a) an excavation wall and (b) drill core from nearby the studied New Mine Level zone (modified after Skewes et al., 2002)

Although the means and single standard deviations are in reasonable agreement with the EDZ limits by Perras and Diederichs (2015), some of the maximum overbreak cases fall above the limits defined for their respective lithologies. New functions for improved predictive ranges of maximum to minimum measured cases of brittle overbreak in the heterogeneous and complex stockwork mafic complex, dacite porphyry, and brecciated units are provided in Table 4-7 and illustrated in Figure 4-27 to account for all observed cases. The mechanistic framework of the functions developed by Perras and Diederichs (2015) has been preserved here, where the minimum case (zero) depth of spalling occurs at or below maximum tunnel stresses equal to crack initiation (CI).

Table 4-7: Maximum and minimum predictive functions for heterogeneous complex rockmasses based on observed brittle overbreak at the El Teniente New Mine Level Project

<table>
<thead>
<tr>
<th>Lithology</th>
<th>Maximum Predicted Overbreak</th>
<th>Minimum Predicted Overbreak</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stockwork Mafic Complex</td>
<td>$r / a = 1 + 0.95(\sigma_{\text{max}} / CI - 1)^{0.55}$</td>
<td>$r / a = 1 + 0.65(\sigma_{\text{max}} / CI - 1)^{0.5}$</td>
</tr>
<tr>
<td>Dacite Porphyry</td>
<td>$r / a = 1 + 0.85(\sigma_{\text{max}} / CI - 1)^{0.2}$</td>
<td>$r / a = 1 + 0.5(\sigma_{\text{max}} / CI - 1)^{0.4}$</td>
</tr>
<tr>
<td>Brecciated Units</td>
<td>$r / a = 1 + 0.35(\sigma_{\text{max}} / CI - 1)^{0.35}$</td>
<td>$r / a = 1 + 0.17(\sigma_{\text{max}} / CI - 1)^{0.65}$</td>
</tr>
</tbody>
</table>
Figure 4-27: Ranges of overbreak depths for case drifts A and B at the El Teniente New Mine Level Project, including new failure prediction ranges for these heterogeneous rockmasses defined by functions listed in Table 4-7. The range of “Brecciated Units” includes the anhydrite breccia and the brecciated stockwork mafic complex – dacite porphyry contact units. The empirical linear fit by Diederichs (2010) and after Martin et al. (1999) is included for reference.

Explanations of the significant range between the mean and maximum overbreak cases for each unit are as follows. The stockwork mafic complex has the largest difference between the mean and maximum cases. In the context of measurements along drifts A and B, there are only five measured profiles with overbreak between 2 and 3 m (the maximum overbreak is 2.3 m and 3 m in drifts A and B, respectively). The infrequent occurrence of these maximum cases is attributed to the complex geometry of the vein network, and these maximum overbreak cases are a result of the worst-case alignment of
multiple vein structures that efficiently promotes crack propagation. This stress-driven structural failure mode (as per van der Pouw Kraan, 2014) was observed by Diederichs et al. (2013) in rockmasses with widely spaced but persistent joints. Diederichs et al. (2013) observed extreme cases of failure depth in brittle conditions, where the stress fractures that formed parallel to the excavation boundary migrated up a tapered chimney defined by long and persistent joints (Figure 4-28). A similar appropriate confluence of large-scale planar quartz veins in the stockwork mafic complex at El Teniente explains the extreme but infrequent cases of maximum overbreak.

Figure 4-28: Stress-driven structurally controlled failure where stress fractures accumulate parallel to the excavation boundary and migrate along widely spaced, persistent joints. The result is an extremely deep failure not otherwise found in massive spalling notch formation (Diederichs et al., 2013)

The poor fit of maximum overbreak in the dacite porphyry to the EDZ1 is attributed to the wide variety of alteration minerals present in this unit and their range of strength, including silica, biotite, chlorite, anhydrite, sericite, and clays. The confluence of locally dense stockwork with different mineralogies would increase the amount of overbreak.

In the case of the anhydrite breccia and contact zone breccia, maximum overbreak cases above the HDZ limit are explained by the presence of variable clast sizes in the units. In zones with smaller or infrequent clasts, there are smaller or fewer obstacles to suppress crack propagation, so overbreak in these cases would behave more like a homogeneous massive rock (and therefore have a better correlation to the EDZ1 limit).
4.6 Discussion and Conclusions

Overbreak observations and measurements in the undercut level of the New Mine Level Project at the El Teniente porphyry mine in Chile have been tested here against empirical and mechanics-based predictive tools for depth of failure in brittle conditions. The heterogeneous rockmasses examined here in the undercut level do not correlate to linear empirical trends of overbreak depth for different stress conditions by Martin et al. (1999) and Diederichs (2007).

Further developments of these predictive tools using the mechanistic damage initiation – spalling limit brittle failure criterion approach developed by Diederichs (2007) suggest a more appropriate nonlinear trend of overbreak depth for different stress conditions (Diederichs, 2007; Perras and Diederichs, 2015). The investigation by Perras and Diederichs (2015) was driven by research for nuclear waste disposal, which is interested in individual excavation damage zones (EDZs). The highly damaged zone (HDZ) forms closest to the boundary and is characterized by instantaneous crack propagation and notch formation after initiation. The HDZ transitions to the inner EDZ (EDZi) which exhibits systematic and dilating damage, and will form a notch geometry with aggressive scaling. The EDZi corresponds to the case histories based on maximum damage depth and empirical fit by Martin et al. (1999) and Diederichs (2007). The EDZi transitions to the outer EDZ (EDZo) which is characterized by partial connected to isolated damage and does not have significant dilation. The EDZo is the limit of damage initiation which is normally minor in homogeneous rocks.

The comparison of all three EDZ overbreak limits to the complex rock units at El Teniente in this investigation resulted in an improved fit to the mechanistic overbreak prediction tool for the mean and one standard deviation cases. The pervasive, planar quartz veins in the stockwork mafic complex act as crack attractors and allow for maximum crack propagation after initiation and immediate strength loss. This behaviour has the best correlation to the EDZo limit for homogeneous rocks. The dacite porphyry unit has fewer stockwork veins than the stockwork mafic complex and the mean measured overbreak correlates well to the EDZi limit for homogeneous rocks, which is the most likely limit of overbreak given time, no support, and aggressive scaling for average homogenous brittle rockmasses. The anhydrite
breccia unit contains large clasts of contrasting mineralogy in the intact anhydrite and quartz matrix that act as crack arresters. The contact zone between the stockwork mafic complex and dacite porphyry shows similar overbreak patterns when compared to the anhydrite breccia since this contact zone in the investigated drift was likely mechanically brecciated by the intrusion of the dacite porphyry. These breccia units correlate well to the HDZ limit for homogenous rocks that only includes the immediate rupture after crack initiation.

For the examined maximum overbreak cases in each lithological unit, functions of maximum to minimum ranges of overbreak for these heterogeneous complex rockmasses have been developed to improve predictions of overbreak. These functions preserve the mechanistic framework presented by Perras and Diederichs (2015) while adapting the application from EDZ prediction to heterogeneous complex rockmasses.
4.7 References


Chapter 5

Component and System Deformation Properties of Complex Rockmasses with Healed Structure

5.1 Abstract
The overall strength of a rockmass is determined by the intact rock strength and characteristics of the rockmass structure. In complex rockmasses, this structure consists of both joints and other fractures (interblock structure) and intrablock structure such as veins. As excavations go deeper, intrablock structure has been found to have a significant impact on rockmass behaviour. Complex numerical modelling for modern geotechnical design requires input parameters for structure, including normal and shear stiffness, and strength, which have a critical influence on modelled rockmass behaviour. This study focusses on extending joint stiffness and strength concepts to veins by calibrating numerical finite element simulations of Unconfined Compressive Strength (UCS) tests with explicit vein geometries that were determined by petrographic analysis of veins in thin section. In some cases, different calibrated properties were found to have equivalent stress-strain profiles in UCS tests. The behaviour of the calibrated stiffness and strength vein properties are examined at an excavation scale using a numerical example of a 10 m-diameter tunnel.

5.2 Introduction
The overall strength of a rockmass is determined by the intact rock strength and characteristics of the rockmass structure. Empirical rockmass classification systems quantify components that contribute to

---

rockmass strength, including intact rock strength, joint length, spacing, orientation, shape, aperture and surface condition, into single values that can be used for ground support design. The development of numerical modelling tools has allowed for more complex analyses for modern geotechnical design than empirical classification systems can provide.

As numerical methods grew from their early homogeneous continuum behaviour to including explicit joint elements, and to having fully discrete capabilities, additional input parameters have become necessary to describe the components of rockmass behaviour. In particular, for capturing the behaviour of rockmass structure, the concept of normal and shear stiffness for joint elements was developed to describe the stress-deformation response before yield and sliding occurs (Goodman et al., 1968).

During the early considerations of joint stiffness, healed structures such as veins, veinlets and stockwork that are found in complex rockmasses (termed intrablock structure where joints are interblock structure) were disregarded as having an inconsequential influence on rockmass behaviour (Goodman et al., 1968). However, as both civil and mining underground excavations go deeper and enter into more high stress environments with complex excavation geometries and associated stress paths, healed structures have been found to influence rockmass behaviour.

For complex rockmasses, the conventional theory of joint stiffness must be extended to accommodate strong, brittle infilling materials that compose intrablock structure. Normal stiffness and shear stiffness values for interblock structure are commonly determined by laboratory analysis of Unconfined Compressive Strength (UCS), triaxial, or direct shear tests. Using detailed numerical models for this study, this approach is extended and applied to a variety of samples from a porphyry copper deposit in northern Chile that contain hydrothermal vein intrablock structure.

Petrographic analyses of the vein samples in thin section were conducted to determine the modal mineralogy and geometry of the vein minerals. The veins are first explicitly modelled using material zones to represent individual mineral clusters and/or grains, which require intact material stiffness (Young’s modulus and Poisson’s ratio) and strength (Mohr-Coulomb criterion) properties.
UCS models that contain a numerical joint element (used to represent a vein) are then used to calibrate the joint element properties (normal stiffness, shear stiffness, and Mohr-Coulomb strength) to mimic the stress-strain behaviour of the models where the veins are explicitly modelled with material zones. The behaviour of the calibrated stiffness and strength vein properties are finally examined at an excavation scale using a numerical example of a 10 m-diameter tunnel, in order to analyse the vein behaviour in a system with different vein orientations and loading directions.

5.3 Conventional Consideration of Joint Stiffness

Rockmass behaviour depends on properties of the intact rock and discontinuities such as joints. Joint properties, including normal and shear stiffness, shear and tensile strength, spacing, persistence, aperture, and dilation, have a significant influence on rockmass behaviour. The overall system behaviour can be very sensitive to any or all of these interrelated components. The importance of joint stiffness was recognized more recently than strength properties, with the introduction of finite element (FEM) and finite difference (FDM) numerical methods (Goodman et al., 1968).

The earliest work performed to consider joint stiffness separately from rockmass stiffness was by Goodman et al. (1968), when numerical analyses were beginning to extend beyond continuum analytical conventions, and model joints as discrete features. Joint stiffness is separated into normal stiffness \(K_n\) and shear stiffness \(K_s\), which describe the rate of change of normal stress \(\sigma_n\) and shear stress \(\tau\), respectively, with respect to normal displacement \(\delta_n\) and shear displacement \(\delta_s\). It is important to understand that the deformation characteristics of a joint have no bearing on the failure mode (e.g. tensile, shear) of a joint.
The most important factors that influence joint normal stiffness are (after Goodman et al. 1968):

i. The contact area between the two joint walls, as a percentage of the total area;

ii. The perpendicular aperture distribution and amplitude; and

iii. The relevant properties of the filling materials (if present).

Joint shear stiffness depends on:

i. The roughness of the joint walls as determined by the distribution, amplitude, and inclination of the asperities;

ii. The tangential aperture distribution and amplitude; and

iii. The relevant properties of the filling materials (if present).

For example, clay infilling will indicate low stiffness values; however, while mineral coatings such as chlorite, talc, or graphite on the joints will reduce the joint strength, the stiffness may be high.

5.3.1 Laboratory Testing

There are two main testing options available to measure joint stiffness: UCS/triaxial and direct shear testing. Both techniques have been successful at measuring joint stiffness in numerous testing campaigns. It is important to measure the displacements directly on the sample (instead of on the loading system) and as close as possible to the joint plane so as to minimize discrepancies between the methods (Rosso, 1976). Discrepancies in stiffness results between the test methods can also be explained by the different stress paths during the tests. During direct shear tests, $\sigma_n$ is held constant through the test, while in the triaxial tests, $\sigma_n$ increases with $\tau$, until slip on the joint occurs (Rosso, 1976).

Direct shear testing can be performed at both laboratory and in-situ scales, while UCS and triaxial tests are limited to the scale of drill core. Relatively few in situ tests have ever been performed due to the high cost and difficult setup required. Most in situ direct shear tests have been purposefully conducted on key low strength joints in a given project because they were expected to have the most significant effect on reducing the rockmass strength (Krsmanovic and Popovic, 1966). Therefore, in situ testing data for clean joints is scarce in the literature.
5.3.2 Stiffness Related to Joint Characteristics

The normal and shear stiffness values of a joint are determined using test results in stress-displacement space, in each of the normal and shear directions. The characteristics of the shear stiffness curves at the yield, peak, and ultimate stress-displacement locations vary between different types of joint surfaces. Typical shear stress-displacement profiles are shown in Figure 5-1, from (Goodman, 1969).

![Figure 5-1: Typical shear stress-deformation relationships for various types of interblock structures (after Goodman, 1969)](image)

The normal stiffness of a joint is determined by measuring joint closure under increasing normal stress; however, it cannot be defined by a single value. Instead, for each increment of normal stress, the corresponding normal stiffness is calculated using the incremental change of joint closure (Bandis et al., 1983). Experimental results have shown that this relationship can vary in a near hyperbolic fashion (e.g. Goodman, 1976; Bandis et al., 1983), with hysteretic recovery upon unloading (Bandis et al., 1983). In
some cases at low normal stresses, this relationship has also been found to vary linearly (Hungr and Coates, 1978). An example of normal deformation profiles in a granodiorite sample, considering the intact, mated-joint, and non-mated joint stiffnesses, is shown in Figure 5-2 (from Goodman, 1976). The normal stiffness of a joint increases as closure progresses with increasing applied normal stress. As the maximum closure is reached at a high normal stress, the deformation of the sample becomes increasingly dependent on the stiffness properties of the intact rock (Young’s modulus, $E_i$). The softer behaviour of the non-mating joint is a result of normal stress concentrations over a lower actual contact area. In general, the normal stiffness is larger if the rock wall and infilling material (if present) are stronger and stiffer (Read and Stacey, 2009).

Figure 5-2: Normal compression of an extension fracture in a granodiorite sample, where $V_{mc}$ is the maximum possible closure, and $P$ shows the yield point of the sample, showing a hyperbolic behaviour. The difference between compression curves A and B describes the compression of the mated joint (independent of the system stiffness), as shown in the right graph, and likewise for curves A and C for the non-mated joint (Goodman, 1976)
5.4 Extension of Joint Stiffness to Intrablock Structure

The most logical approach to incorporating intrablock structure into explicit FEM numerical models is to use numerical joint elements with appropriate stiffness and strength values. As such, the stiffness and strength properties of intrablock structure should follow similar logic in testing and application of properties used to model joints and other interblock structure. In general, intrablock structure is expected to be stiffer than interblock structure in both the normal and shear directions, but the behaviour will significantly depend on the mineralogical configuration and properties of the healed infilling material. The numerical study presented in this paper is a first attempt to quantify stiffness and strength properties for intrablock structure by considering petrographic analysis of rock samples in numerical models.

5.5 Petrographic Assessment of Intrablock Structure

Petrographic analyses of veins (intrablock structure) in thin section were conducted to determine the modal mineralogies and mineral grain geometries of the veins for input to geomechanical numerical models. Hand samples of drill core with veins of a variety of lithologies were collected from a porphyry copper ore deposit in northern Chile and cut into thin sections. Of the 28 thin section samples, four were selected for this study, based on a variety of lithologies in the veins, relatively large grain sizes (for accurate petrographic identification), and clear boundaries between the vein and wall rock. Petrographic analysis of the thin sections was conducted to determine the modal mineralogies of the veins (see Table 5-1). The behaviour characteristics of the vein minerals are shown in Table 5-2.

Table 5-1: Modal mineralogies of the vein samples

<table>
<thead>
<tr>
<th>Minerals</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quartz</td>
<td>95%</td>
<td>-</td>
<td>-</td>
<td>3%</td>
</tr>
<tr>
<td>Pyrite</td>
<td>5%</td>
<td>2%</td>
<td>-</td>
<td>60%</td>
</tr>
<tr>
<td>Calcite</td>
<td>-</td>
<td>85%</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Gypsum</td>
<td>-</td>
<td>-</td>
<td>100%</td>
<td>-</td>
</tr>
<tr>
<td>Muscovite</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>37%</td>
</tr>
<tr>
<td>Wall rock inclusions</td>
<td>-</td>
<td>13%</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
The Mohs hardness ratings and tenacity descriptions of the minerals indicate relative stiffness and/or strength behaviour of the minerals. Photos of the four hand samples used in this study are shown in Figure 5-3, and composite photos of the veins in the corresponding thin sections are shown in Figure 5-4.

**Table 5-2: Behaviour characteristics of the vein minerals**

<table>
<thead>
<tr>
<th>Minerals</th>
<th>Mohs Hardness</th>
<th>Tenacity</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quartz</td>
<td>7</td>
<td>Brittle</td>
<td>Nesse, 1991</td>
</tr>
<tr>
<td>Pyrite</td>
<td>6-6.5</td>
<td>Brittle</td>
<td>Nesse, 1991</td>
</tr>
<tr>
<td>Calcite</td>
<td>3</td>
<td>Ductile</td>
<td>Nesse, 1991</td>
</tr>
<tr>
<td>Gypsum</td>
<td>2</td>
<td>Flexible</td>
<td>Nesse, 1991</td>
</tr>
<tr>
<td>Muscovite</td>
<td>2.5-3</td>
<td>Flexible/ elastic/ ductile</td>
<td>Nesse, 1991</td>
</tr>
</tbody>
</table>

Figure 5-3: Photos of hand samples (1 to 4) selected for thin section analysis and use in this study
Figure 5-4: Composite photos of veins (samples 1 to 4) in cross-polarized, transmitted light thin section. The length of all scale bars represents 5 mm in the thin sections
5.6 Effects of Joint Stiffness in Numerical Models

To determine stiffness and strength values for the four vein samples, numerical analyses of UCS laboratory tests were conducted using Phase², a FEM software (RocScience, 2013). It is important to note that these simulations are in two-dimensional software, which does not account for three-dimensional effects that may be encountered in real laboratory UCS testing. However, this study is focussed on the effects of the cross-cutting intrablock structures and a two-dimensional analysis is required for the implementation of thin sections of vein materials. Further analysis using three dimensional numerical simulations is a recommended topic of future research.

Explicit models were built, where the vein grain and/or cluster boundaries were represented by numerical material boundaries, using intact stiffness (Young’s modulus and Poisson’s ratio) and strength (Mohr-Coulomb tensile strength, cohesion, and friction angle) values from the literature for the individual minerals. Corresponding UCS samples were built where the veins were represented by numerical joint elements, and the joint element normal and shear stiffness (Kₙ and Kₛ), and strength (Mohr-Coulomb criterion) values were calibrated to have equivalent stress-strain behaviour as the explicit models with mineral zones.

5.6.1 Model Setup

The intact strength properties for the wall rock material were selected to represent a partially leached andesitic tuff with a Poisson’s ratio of 0.3. The effect of Young’s modulus in the wall rock was analysed by comparing samples of 37,500 MPa and 75,000 MPa for all four vein types. Using the Generalized Hoek-Brown criterion (as per Hoek et al., 2002), the intact compressive rock strength (σᵢ) is 70 MPa, the mᵢ value is 16, and the Geological Strength Index (GSI) value is 100. Residual intact wall rock properties are based on a GSI of 65. A schematic of the general UCS model geometry and boundary conditions is shown in Figure 5-5.
Figure 5-5: Schematic of FEM models of UCS test simulations showing geometry, boundary, and loading conditions, for the two methods of modelling veins: by explicit vein materials and a calibrated, equivalent numerical joint element.
The explicit mineral zones and joint element veins are oriented at 30° from the loading axis, and they are shown together in Figure 5-5 for descriptive purposes only. The vein orientation is selected to approximate predicted critical failure angles of this UCS sample and to ensure the veins daylight on the circumference of the sample without interference from the steel platens.

The intact stiffness values for individual minerals are shown in Table 5-3. Mineral strength criteria are selected based on the tenacity descriptions associated with each mineral, which indicate the physical reaction of a mineral to applied stresses (as shown in Table 5-2). Minerals with a brittle tenacity are represented by the cohesion loss and friction mobilization strength model (e.g. Martin, 1997), while minerals with flexible, elastic, and/or ductile tenacities are represented by the conventional strain-softening Mohr-Coulomb strength criterion. The mineral strength properties are shown in Table 5-4.

**Table 5-3: Elastic properties of vein minerals**

<table>
<thead>
<tr>
<th>Minerals</th>
<th>Young’s modulus (MPa)</th>
<th>Poisson’s ratio</th>
<th>References</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quartz</td>
<td>70,060</td>
<td>0.19</td>
<td>Lama and Vutukuri, 1978; Li et al., 2003; Carmichael, 1982; Birch, 1966</td>
</tr>
<tr>
<td>Pyrite</td>
<td>227,700</td>
<td>0.17</td>
<td>Lama and Vutukuri, 1978; Birch, 1966</td>
</tr>
<tr>
<td>Calcite</td>
<td>89,500</td>
<td>0.29</td>
<td>Lama and Vutukuri, 1978; Birch, 1966; Holbrook, 2000</td>
</tr>
<tr>
<td>Gypsum</td>
<td>35,300</td>
<td>0.34</td>
<td>Robertson et al., 1958</td>
</tr>
<tr>
<td>Muscovite</td>
<td>56,875</td>
<td>0.25</td>
<td>Lama and Vutukuri, 1978; Birch, 1966</td>
</tr>
</tbody>
</table>

The explicit vein models in Phase² employ material boundaries to divide the veins into zones of individual materials that represent mineral grains and/or clusters. Using material boundaries in Phase² software is a simple, effective approach since material boundaries are polylines without any associated behaviour properties (RocScience, 2013). Detailed views of the veins in Phase² and the corresponding composite photos of thin sections are shown in Figure 5-6. The density and gradation of the material mesh are also visible in the figure. The thicknesses of the veins are approximately 10 mm (sample 1), 4 mm (sample 2), 8 mm (sample 3), and 15 mm (sample 4).
Figure 5-6: Composite photos of veins and corresponding FEM explicit vein models in Phase²
Table 5-4: Strength properties of vein minerals

<table>
<thead>
<tr>
<th>Mineral</th>
<th>PEAK</th>
<th>RESIDUAL</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>σt (MPa)</td>
<td>c (MPa)</td>
</tr>
<tr>
<td>Quartz (brittle)</td>
<td>2</td>
<td>20</td>
</tr>
<tr>
<td>(Lama and Vutukuri, 1978; Li et al., 2003; Laws et al., 2003)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pyrite (brittle)</td>
<td>2</td>
<td>4.7</td>
</tr>
<tr>
<td>(Lama and Vutukuri, 1978)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Calcite (brittle)</td>
<td>0.5</td>
<td>1.5</td>
</tr>
<tr>
<td>(Lama and Vutukuri, 1978)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gypsum (Robertson et al., 1958)</td>
<td>2.2</td>
<td>16.6</td>
</tr>
<tr>
<td>Muscovite</td>
<td>1.5</td>
<td>5</td>
</tr>
<tr>
<td>(Lama and Vutukuri, 1978)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

5.6.2 FEM Numerical Model Results

Several observations can be made from the models that represent each of the vein samples. Results of the UCS tests are shown in axial stress-strain space in Figure 5-7, including profiles of the explicitly modelled veins (to which the samples with joint elements were calibrated). The behaviour between models with intact Young’s moduli values of 37,500 and 75,000 MPa for the wall rock are very dependent on the Young’s modulus values for the individual vein minerals, which results in significant changes in the axial stress-strain behaviour.

Vein stiffness and strength properties for UCS samples with numerical joint elements were calibrated to the samples with explicitly modelled veins. The calibrated stiffness properties are shown in Table 5-5, and the strength properties are shown in Table 5-6. It is interesting to note that, in cases with relatively soft vein behaviour, it is possible to have multiple combinations of normal and shear stiffness values and ratios that produce the same stress-strain behaviour in the UCS test (e.g. samples 2 and 4). However, it is hypothesized that at an excavation scale with different vein orientations and loading directions, not all observations at the UCS sample scale (with a single vein orientation and loading direction) would be applicable. The excavation scale behaviour will be discussed further in the following section.
During calibration of the Mohr-Coulomb vein strength properties, it was observed that cohesion (c) had the most significant influence on the yield strength, followed by tensile strength (σt), and friction angle (ϕ) had the least influence.

Figure 5-7: Stress-strain profiles of UCS tests of each sample (1 to 4) with both wall rock Young’s moduli values (37,500 and 75,000 MPa). The vein samples with real modal mineralogies are compared to samples of veins with 100% of each mineral present, in order to compare the relative influence of minerals on the overall axial stress-strain behaviour. Due to the ideal nature of numerical models, the pre-peak elastic portions of the curves are linear.
Table 5-5: Calibrated vein stiffness properties. The highlighted rows are used to easily distinguish between the different intact Young’s Modulus (\(E_i\)) cases.

<table>
<thead>
<tr>
<th>Model No.</th>
<th>(E_i) (MPa)</th>
<th>(K_n) (MPa/m)</th>
<th>(K_s) (MPa/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. sample 1</td>
<td>37,500</td>
<td>6,000,000</td>
<td>6,000,000</td>
</tr>
<tr>
<td>2. sample 1</td>
<td>75,000</td>
<td>7,000,000</td>
<td>7,000,000</td>
</tr>
<tr>
<td>3. sample 2</td>
<td>37,500</td>
<td>13,500</td>
<td>13,500</td>
</tr>
<tr>
<td>4. sample 2</td>
<td>37,500</td>
<td>100,000</td>
<td>9500</td>
</tr>
<tr>
<td>5. sample 2</td>
<td>75,000</td>
<td>11,500</td>
<td>11,500</td>
</tr>
<tr>
<td>6. sample 2</td>
<td>75,000</td>
<td>100,000</td>
<td>9000</td>
</tr>
<tr>
<td>7. sample 3</td>
<td>37,500</td>
<td>5,000,000</td>
<td>5,000,000</td>
</tr>
<tr>
<td>8. sample 3</td>
<td>75,000</td>
<td>4,000,000</td>
<td>4,000,000</td>
</tr>
<tr>
<td>9. sample 4</td>
<td>37,500</td>
<td>5,000,000</td>
<td>5,000,000</td>
</tr>
<tr>
<td>10. sample 4</td>
<td>75,000</td>
<td>40,000</td>
<td>4000</td>
</tr>
<tr>
<td>11. sample 4</td>
<td>75,000</td>
<td>5300</td>
<td>5300</td>
</tr>
<tr>
<td>12. sample 4</td>
<td>75,000</td>
<td>10,000</td>
<td>4500</td>
</tr>
</tbody>
</table>

Table 5-6: Calibrated Mohr-Coulomb vein strength properties. The highlighted rows are used to easily distinguish between the different intact Young’s Modulus (\(E_i\)) cases.

<table>
<thead>
<tr>
<th>Model No.</th>
<th>PEAK</th>
<th>RESIDUAL</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(\sigma_t) (MPa)</td>
<td>(c) (MPa)</td>
</tr>
<tr>
<td>1. sample 1</td>
<td>0.1</td>
<td>4.5</td>
</tr>
<tr>
<td>2. sample 1</td>
<td>0.1</td>
<td>4.1</td>
</tr>
<tr>
<td>3. sample 2</td>
<td>0.1</td>
<td>0.08</td>
</tr>
<tr>
<td>4. sample 2</td>
<td>0.1</td>
<td>0.08</td>
</tr>
<tr>
<td>5. sample 2</td>
<td>0.1</td>
<td>0.08</td>
</tr>
<tr>
<td>6. sample 2</td>
<td>0.1</td>
<td>0.08</td>
</tr>
<tr>
<td>7. sample 3</td>
<td>0.1</td>
<td>17.5</td>
</tr>
<tr>
<td>8. sample 3</td>
<td>0.1</td>
<td>7</td>
</tr>
<tr>
<td>9. sample 4</td>
<td>0.1</td>
<td>1.1</td>
</tr>
<tr>
<td>10. sample 4</td>
<td>0.1</td>
<td>0.025</td>
</tr>
<tr>
<td>11. sample 4</td>
<td>0.1</td>
<td>0.025</td>
</tr>
<tr>
<td>12. sample 4</td>
<td>0.1</td>
<td>0.025</td>
</tr>
</tbody>
</table>
In order to determine the relative contribution of each vein mineral to the stress-strain behaviour of the whole vein, UCS models were created with a single vein mineral applied to the vein material zone. These axial stress-strain results are shown in Figure 5-7 alongside the “real” explicit vein stress-strain profiles with the true modal mineralogy of the samples.

In sample 1, the stiffness of the 100% quartz and 100% pyrite veins are similar (as reflected by the real vein models), but the yield strengths vary. Although the vein is 95% quartz and 5% pyrite, the strength of pyrite has a significant influence on the yield strength of the vein. In sample 2, the stress-strain behaviour of the real vein is controlled by the calcite content (85% abundance). In the UCS model, all of the calcite yields but the wall rock inclusions and pyrite grains remain intact. In sample 3, the real vein is 100% gypsum so there is no applicable comparison between minerals.

In sample 4, the real vein is both softer and weaker than the individual minerals overall. This is an unexpected observation when only the modal mineralogy is considered (60% pyrite, 37% muscovite, and 3% quartz). Since the majority of the vein composition is relatively stiff and strong, the real vein would be expected to have a stress-strain profile at least stiffer and stronger than the 100% muscovite vein sample. However, this example shows the importance of mineral grain geometry in the veins relative to the loading direction and the effect on behaviour. In this vein, the muscovite grains occur along the entire length of the wall rock interface, while the pyrite and quartz grains are concentrated in the middle of the vein. In the UCS test, the upper layer of muscovite (~1/3 of the total vein thickness) in the vein yields first (part (i) in Figure 5-8), then (ii) the muscovite lenses in between the pyrite grain boundaries through the middle of the vein yield, followed by (iii) the lower layer of muscovite and finally (iv) some of the pyrite and quartz grains.
Figure 5-8: Yield progression of the explicitly modelled sample 4 vein, with a Young’s modulus value of 75,000 MPa, that contains 60% pyrite, 37% muscovite and 3% quartz.
5.7 Implications for Modelling at an Excavation Scale

To test the design implications of using different Young’s moduli for intact rock and different normal and shear stiffnesses for vein input parameters, a numerical analysis of a circular tunnel was conducted. Numerical models of a 10 m-diameter tunnel were created using FEM Phase² software (RocScience, 2013). The tunnel was tested for each case of the calibrated results from the UCS sample scale.

5.7.1 Model Setup

The tunnel example is designed to be approximately 600 m deep and in an in situ stress state with a K ratio of 1.5:1 (H:V), such that \( \sigma_1 = 24 \text{ MPa} \) (horizontal and perpendicular to the tunnel axis), \( \sigma_2 = 20 \text{ MPa} \) (horizontal and parallel to the tunnel axis), and \( \sigma_3 = 16 \text{ MPa} \) (vertical). The intact lab strength properties for the rock material are the same as the wall rock material used in the UCS sample models. The calibrated vein stiffness and strength properties are applied to the explicit vein networks (see Table 5-5 and Table 5-6). Like the UCS models, models sets with two intact Young’s modulus values, 37,500 MPa and 75,000 MPa, are used to compare their relative influence on rockmass behaviour at an excavation scale.

A diagram of the general model geometry and boundary conditions for all tunnel models is shown in Figure 5-9. The GSI values in the homogeneous equivalent continua are all 70, based on previous work by the author to represent intrablock structure with GSI (e.g. Day et al., 2013). The equivalent continuum sections of the models are used to reduce the model computation time. The relative distances between the tunnel, discrete and continuum zones in Figure 5-9 are approximately to scale.

The veins are modelled explicitly using Voronoi joint element networks, which create polygonal shapes with high angularity and in an irregular pattern, and are widely accepted for modelling microstructures in rock (Lan et al., 2010). An average spacing of 20 cm was used. Geometrical transformation of the Voronoi networks in Phase² was performed in order to create anisotropy in the vein orientation at an apparent dip of 35°. The anisotropic vein network is used to reflect specific orientations of vein sets identified from stereonet analysis of drill core data from the same ore deposit as the samples used in this study (Day et al., 2014). A detailed view of the anisotropic Voronoi vein network is shown in Figure 5-10.
Figure 5-9: Diagram of the general model geometry used for all tunnel models

Figure 5-10: Detailed view of the Phase² FEM tunnel model showing the anisotropic Voronoi vein network
5.7.2 Model Results

The excavation scale models show important differences in rockmass behaviour between each comparison of Young’s modulus of the intact rock and stiffness and strength values of the vein networks. Percentages of yielded joint elements for each model are shown in Table 5-7. Detailed views of the model results, in terms of the major principal stress (σ₁), total displacement, and yielded mesh elements are shown in Figure 5-11 and Figure 5-12. Observations for tunnel models with each vein sample type are as follows:

Vein sample 1 (models 1-2): The vein stiffness and strength values between models with different intact Young’s moduli are very similar since the stiffnesses of both vein minerals (quartz and pyrite) in both cases are greater than that of the wall rock. This is reflected by similar stresses, displacements, yielded elements, and the amount of yielded joint elements in the tunnel models. The vein stiffnesses were increased to extremely high levels (relative to conventional joint stiffness magnitudes) during the calibration process, and multiple combinations of \( K_n \) and \( K_s \) were not found.

Vein sample 2 (models 3-6): Two combinations of \( K_n \) and \( K_s \) were determined for each of the two intact Young’s moduli models, with \( K_n:K_s \) ratios of 1:1 and ~10:1. The 1:1 vein stiffness ratio at the tunnel scale resulted in much greater total displacements around the excavation (up to 1.5 m) and more yielded elements. However, the veins with a ~10:1 stiffness ratio had a larger percentage of yielded veins, where most of the additional yielded veins were oriented perpendicular to the elongated axis of the vein anisotropy.

Vein sample 3 (models 7-8): These results are very similar to those in vein sample 1, except for a greater difference in the amount of yielded joint elements between models with different Young’s moduli.

Vein sample 4 (models 9-12): The greatest differences between tunnel models occur in the tunnel models for vein sample 4. There is a significant difference in vein stiffness and even strength values between the models with an intact Young’s modulus of 37,500 MPa and 75,000 MPa, which corresponds to significant differences in rockmass behaviour at the tunnel scale. In the \( E_i = 37,500 \) MPa model, there is a more elevated induced stress in the roof and floor of the tunnel, but less total displacement (due to a
smaller percentage of yielded vein elements) and less material element yield than that of the $E_i = 75,000$ MPa model. Between the three $E_i = 75,000$ MPa models with different vein stiffnesses (but the same strengths), the $K_n:K_s$ ratios with the same stress-strain behaviour at the UCS scale range from 10:1 to ~2:1 to 1:1. Where $K_n:K_s$ is 10:1, there are twice the number of yielded vein elements compared to the other two models, which have the same amount of yielded vein elements. Similar to model 4 (vein sample 2), the majority of additional yielded vein elements are oriented perpendicular to the vein anisotropy. Models 11 and 12 (where $K_n:K_s$ is 1:1 and ~2:1, respectively) show elevated levels of stress ($\sigma_1$) in lenses between veins away from the excavation, near the outer limits of the vein yield zone. In these two cases, which have the lowest vein stiffness values of all of the models, the soft veins allow the intact rock to accommodate more induced stress before failure than is seen in any other case.

Regarding the relative impact of vein mineralogy on the rockmass behaviour, the quartz and pyrite veins (sample 1) and the gypsum veins (sample 3) experienced the least amount of failure around the tunnel. The calcite in sample 2 and muscovite in sample 4 were detrimental to the strength of the veins and the overall rockmass behaviour.

Table 5-7: Quantitative numerical results of the amount of vein yield the tunnel models

<table>
<thead>
<tr>
<th>Model No.</th>
<th>Young's Modulus, $E_i$ (MPa)</th>
<th>% Yielded Vein Elements</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. sample 1</td>
<td>37,500</td>
<td>5%</td>
</tr>
<tr>
<td>2. sample 1</td>
<td>75,000</td>
<td>5%</td>
</tr>
<tr>
<td>3. sample 2</td>
<td>37,500</td>
<td>15%</td>
</tr>
<tr>
<td>4. sample 2</td>
<td>37,500</td>
<td>22%</td>
</tr>
<tr>
<td>5. sample 2</td>
<td>75,000</td>
<td>17%</td>
</tr>
<tr>
<td>6. sample 2</td>
<td>75,000</td>
<td>26%</td>
</tr>
<tr>
<td>7. sample 3</td>
<td>37,500</td>
<td>1%</td>
</tr>
<tr>
<td>8. sample 3</td>
<td>75,000</td>
<td>4%</td>
</tr>
<tr>
<td>9. sample 4</td>
<td>37,500</td>
<td>8%</td>
</tr>
<tr>
<td>10. sample 4</td>
<td>75,000</td>
<td>20%</td>
</tr>
<tr>
<td>11. sample 4</td>
<td>75,000</td>
<td>10%</td>
</tr>
<tr>
<td>12. sample 4</td>
<td>75,000</td>
<td>10%</td>
</tr>
</tbody>
</table>
Figure 5-11: Major principal stress ($\sigma_1$, MPa), total displacement (m), and yielded element results for models 1-6 (samples 1 and 2)
Figure 5-12: Major principal stress ($\sigma_1$, MPa), total displacement (m), and yielded element results for models 7-12 (samples 3 and 4)
5.8 Discussion and Conclusions

The numerically simulated UCS tests of samples with veins and the numerical example of a 10 m-diameter tunnel discussed in this paper show that the intact stiffness (Young’s modulus) and vein (normal and shear) stiffness have a very significant influence on rockmass behaviour in underground excavations. The calibration process to equate joint element stiffness and strength properties to explicitly modelled vein examples, based on petrographic analysis of veins in thin section, is tedious and in some cases results in multiple acceptable values for stiffness and strength that match the axial stress-strain behaviour in a UCS test. It was hypothesized that the equivalent stress-strain profiles were not applicable to an excavation scale where vein orientations and loading directions vary throughout the system.

Results from the example tunnel models support this hypothesis and show that variations in induced stress, total displacement, and the amount and extent of both yielded material elements and explicit vein elements, are possible. However, further investigation is required to determine the correct unique stiffness and strength values for veins at an excavation scale. A starting point for this investigation would be to build UCS models with explicitly modelled veins at several orientations between 0° and 90° to the loading axis, in order to determine stiffness and strength values for veins at a variety of orientations.

The effects of different Young’s modulus values of the wall rock on the vein behaviour are significant. The stress-strain profiles from the UCS samples showed that changes in Young’s modulus in the wall rock resulted in changes in both stiffness and yield strength of the sample. If the Young’s modulus of the vein mineral was initially higher than the 37,500 MPa in the wall rock but then became lower than the 75,000 MPa in the wall rock, such as muscovite, the development of failure in the vein, as well as stiffness and strength values, changed dramatically. Although in some cases there were very stiff and strong minerals present in the veins, the softer and weaker veins tended to control the overall vein behaviour.

Furthermore, there were notable differences in strength and stiffness values between the veins of different modal mineralogies. Study of additional vein thin section samples is needed to test this methodology in a wider variety of vein mineralogies. Beyond numerical studies, research with a focus on
physical laboratory testing, using UCS, triaxial, and direct shear tests on a variety of lithologies and vein mineralogies is necessary to improve the quantification of the stiffness and strength properties of intrablock structure.
5.9 References


Chapter 6

Optimization of Structural Contact Stiffness and Strength for Discrete Simulation of Progressive Failure of Healed Structure

6.1 Abstract
Geotechnical analysis for underground excavation design in complex tectonic environments requires an increased understanding and more rigorous consideration of the impact of healed or “intrablock” structure, such as veins, on rockmass behaviour. Intrablock structure occurs between blocks of rock defined and bounded by “interblock structure”, the network of joints and other fractures conventionally considered in classic rockmass characterization, classification or rockmass property estimation. Discrete simulation of fractures has become a more commonplace model analysis technique for excavations in jointed rockmasses. Here too, however, special attention is required to simulate intrablock structure within the model. In particular, the selection and evolution of stiffness and strength values for the model discontinuity elements must follow a different logic than that adopted for fractures and true joints. A new concept to better represent the behaviour of intrablock structure in explicit numerical models is proposed and tested here using finite element method (FEM) analysis and case study data from a 1200 m deep drift in a hydrothermally altered geological setting. This approach changes the stiffness and strength values of failed intrablock structural elements between pre-peak (“primary”), post-peak (“secondary”), and ultimate (“tertiary”) states. The FEM models in the tertiary state match 96% of overbreak patterns along the case drift, versus 80% in primary state models. These findings suggest that the proposed method is a good option to model the influence of intrablock structure on rockmass behaviour more accurately.

---

4 This chapter is published in an international journal with the following citation:
6.2 Introduction

In this paper, rockmass structure in tectonically complex domains is divided into natural joints or fractures (physical breaks in the rock) that define rock blocks (interblock structure) and healed structures such as veins (intrablock structure) that exist within the blocks bounded by joints. Rockmass stiffness and strength parameters are determined by the characteristics of intact rock and rockmass structure. Empirical rockmass classification systems quantify components that contribute to rockmass strength, including intact rock strength, joint length, spacing, orientation, shape, aperture, and surface condition, into single values that can be used for ground support design. These systems typically consider only joints and physical fractures (interblock structure). Day et al. (2012) have introduced strategies for inclusion of healed intrablock structure into empirical rockmass characterization.

Advancement of numerical modelling tools has allowed for more detailed analysis of rockmass structure in modern geotechnical design than empirical classification systems can provide. As numerical methods grew from their early homogeneous continuum behaviour to include explicit joint elements, and to having fully discrete capabilities, additional input parameters became necessary to describe the rockmass behaviour. In particular, for capturing the behaviour of rockmass structure, the concept of normal and shear stiffness for joint elements was developed to describe the stress-deformation response before yield and sliding occur (Goodman et al., 1968).

During the early considerations of rockmass stiffness and strength, healed structures such as veins, veinlets and stockwork that are found in tectonically complex rockmasses were disregarded as having an inconsequential influence on rockmass behaviour (Goodman et al., 1968). However, as both civil and mining underground excavations go deeper and enter into more high stress environments with complex excavation geometries and associated stress paths, healed structures have been found to have a significant impact on rockmass behaviour. Using case data, this study explores a new concept of primary, secondary, and tertiary values for stiffness and strength, defined by pre-peak, post-peak, and ultimate states of yield, for intrablock structure in complex rockmasses. Finite Element Method (FEM) numerical models of a mining drift with explicit intrablock structure are compared to overbreak profiles of a 1200 m deep drift in an undercut level during the development of a block cave mining operation in Chile.
6.3 Joint Stiffness

The earliest work performed that considers joint stiffness separately from rockmass stiffness is by Goodman et al. (1968), which was conducted when numerical analyses were beginning to extend beyond continuum analytical conventions and to model joints as discrete features. Joint stiffness is separated into components of normal stiffness ($K_n$) and shear stiffness ($K_s$), which describe the rate of change of normal stress ($\sigma_n$) and shear stress ($\tau$), respectively, with respect to normal displacement ($\delta_n$) and shear displacement ($\delta_s$). The most important factors that influence joint stiffness are, for normal stiffness, the contact area between the joint walls, the perpendicular aperture distribution and amplitude, and the filling materials. Shear stiffness depends on the roughness of the joint walls, the tangential aperture distribution and amplitude, and the filling materials.

The normal stiffness of a joint increases as closure progresses with increasing normal stress. As the maximum closure is reached at a high normal stress, the deformation of the sample becomes increasingly dependent on the stiffness properties of the intact rock. For each increment of normal stress, the corresponding normal stiffness is calculated using the incremental change of joint closure (Bandis et al., 1983). Early experimental results suggested that this relationship varies in a near hyperbolic fashion (e.g. Goodman, 1976), with hysteretic recovery upon unloading.

A linear relationship has also been documented for testing at low maximum normal stresses (Hungr and Coates, 1978); however, this would likely develop into nonlinear behaviour at higher normal stresses. Further studies have since modified the hyperbolic closure law to a semi-logarithmic closure law where closure varies as the logarithm of normal stress (Bandis et al., 1983). This closure law has only one rockmass parameter, the “stiffness characteristic”, which can be used to fully describe the normal stiffness behaviour of a fracture at any normal stress (Zangerl et al., 2008).
6.4 Shear Strength of Rough Surfaces

For peak strength, the roughness of naturally occurring joint surfaces in hard rock is defined by undulations and asperities, which have a significant influence on their shear behaviour. In general, larger undulations and asperities increase the roughness of a joint surface, which in turn increases its shear strength. The irregularities also influence the path of shear behaviour, where at low normal stresses, shear displacement occurs as the joint surfaces dilate and ride over the irregularities. At high stresses, shear displacement occurs by exceeding the intact material strength of the wall rock and shearing through the irregularities, with minimal dilation. Bi-linear strength envelopes using the Mohr-Coulomb criterion were first developed to address rough joint shear behaviour that is dependent on normal stress, as shown in Figure 6-1 (e.g. Patton, 1966). Non-linear strength envelopes have been developed because changes in shear behaviour with increasing normal stresses are gradual in reality (e.g. Barton and Choubey, 1977) (see Figure 6-1).

The residual strength value of a joint falls below the peak strength. Residual strength is considered after the shear displacement of a joint continues beyond its peak strength when the asperities and other rough features have been sheared through or ground down to a smooth plane.

![Figure 6-1: Bi-linear (Patton, 1966) and non-linear (Barton and Choubey, 1977) criteria for rough joint shear strength](image_url)

**Bi-linear approach**

Low stress: \( \tau = \sigma_n \tan(\phi_r + i) \)

High stress: \( \tau = c + \sigma_n \tan \phi \)

**Barton-Bandis non-linear approach**

\[
\tau = \sigma_n \tan \left( \phi_r + \frac{JRC \log_{10} \left( \frac{JCS}{\sigma_n} \right)}{s_{\phi}} \right)
\]

\( \phi_r = (\phi_b - 20) + 20(\tau/R) \)

Where:
- \( \phi_b \) = basic friction angle of surface
- \( i \) = angle of asperity
- JRC = joint roughness coefficient
- JCS = joint wall compressive strength
- \( \phi_r \) = residual friction angle
- \( \tau \) = Schmidt rebound, weathered surface
- \( R \) = Schmidt rebound, dry unweathered surfaces

190
6.5 Extension of Joint Stiffness and Strength to Healed Structure

Healed structures such as hydrothermal veins (see an example photo of a quartz vein in Figure 6-2a), veinlets and stockwork, termed intrablock structure (where joints and other natural fractures are interblock structure), are found in complex rockmasses and have been shown to affect rockmass behaviour in high stress environments.

A good understanding of joint stiffness and strength is necessary to incorporate intrablock structure into explicit and discrete detailed numerical models so that such models will be compatible with existing methodologies for numerical modelling of intact rock and interblock structure. In general, intrablock structure is initially expected to be stiffer and stronger than interblock structure; the values will depend on the mineralogical configuration and properties of the healed infilling material, and with some consideration of the wall rock properties.

![Figure 6-2](image)

**Figure 6-2**: (a) Drill core section of wall rock with quartz vein; (b) primary (pre-peak), secondary (post-peak), and tertiary (ultimate) considerations of intrablock structure (e.g. vein) stiffness and strength

The modelling approach presented in this paper introduces and tests the concept of changing stiffness and strength values for intrablock structure between pre-peak (primary), post-peak (secondary), and ultimate (tertiary) states (see Figure 6-2b). In the primary state, when the intrablock structure is intact, the stiffness
and strength properties are related to the intact properties of the infilling material. In the secondary state, immediately after brittle failure, the stiffness and strength values should resemble those of interblock structure with rough and clean surfaces, as features of the new fracture surface in the secondary state will control the subsequent behaviour of the intrablock structure in the rockmass. The tertiary state represents the ultimate fracture condition after shearing, with smooth surfaces and possibly loose infill material. Here, the strength properties resemble those of conventional residual joint strength. The stiffness properties in the tertiary state are assumed to be equal to the secondary state in this study.

**6.5.1 Implementation Method for FEM Software**

In order to transition between the primary, secondary, and tertiary states using the FEM software Phase² (RocScience, 2014), two models (of a mining drift for the case study presented herein) are required. The first model contains vein elements whose peak stiffness and Mohr-Coulomb strength properties represent the primary state; the residual properties represent a transition into the secondary state (a “primary residual” state) where secondary state strength properties are implemented, but primary state stiffness properties remain.

For the vein elements that yield after excavation in the first model, their properties are manually changed for the start of the second model, such that the peak properties represent the secondary state, and the residual properties represent the tertiary state. It should be noted that veins which remain intact after excavation in the first model continue with the same properties in the second model. The first model, with an initial mixture of primary and a transition into secondary (primary residual) state vein properties, is computed again for the second model, with secondary and tertiary state vein properties, to determine the final extent of vein yield. The tertiary state and thus final conditions in the FEM models are reached after computation of the second model. A limitation of this software includes the thresholds between primary, secondary, and tertiary stages are controlled by stresses. Further analysis with more flexible software to implement strain-dependent thresholds is recommended.
6.6 Case Study: Numerical Analysis at the Excavation Scale

To determine the effect of primary, secondary, and tertiary state values of stiffness and strength for intrablock structure, numerical analyses of rockmass behaviour at the excavation scale were conducted using Phase\(^2\) (version 8.020, RocScience, 2014), a Finite Element Method (FEM) software. The excavation scale models were developed using data from a case study of a drift that is located in the undercut level of a developing block caving mine in Chile, and the results are compared to overbreak measurements from the drift. The drift is located at approximately 1200 m depth, where the in-situ principal stresses are \(\sigma_1 = 55\) MPa (trend/plunge: 131/23), \(\sigma_2 = 29\) MPa (248/47), and \(\sigma_3 = 27\) MPa (025/34), which were measured by overcoring near the entrance to the drift shortly before its development. The vertical and horizontal stress components that were used in the models for this study are \(\sigma_v = 32\) MPa, \(\sigma_{h,NS} = 38\) MPa (parallel to drift axis), and \(\sigma_{H,EW} = 42\) MPa (perpendicular to drift axis).

6.6.1 Model Setup

The intact strength properties for the rock material were selected to represent an andesitic porphyry with a Young’s modulus of 51 GPa and a Poisson’s ratio of 0.3. Using the Generalized Hoek-Brown criterion (Hoek et al., 2002) for the intact rock strength, the intact compressive rock strength \((\sigma_i)\) is 160 MPa, the \(m_i\) value is 7, and the Geological Strength Index (GSI) value is 100. Residual intact rock properties are based on a GSI of 65. A schematic of the general model geometry and boundary conditions for all excavation models is shown in Figure 6-3. The relative distances between the excavation, discrete, and continuum zones in Figure 6-3 are approximately to scale. The GSI value in the homogeneous equivalent continuum material is 80 to account for the intrablock structure, and the methodology for this assessment is based on previous work by the author to represent intrablock structure using GSI (the Composite GSI approach; e.g. Day et al., 2013). The equivalent continuum section of the models is used to reduce the model computation time.
The intrablock structure present in the rockmass at the location of the case study is a stockwork of quartz veins, occasionally with other trace minerals. The veins are modelled explicitly using four sets of joint element networks (parallel, variably spaced and non-persistent). The general spacing and orientations of the vein network are based on current observations at a drift heading approximately 100 m away from, and in the same orientation as the historical case study drift (see Figure 6-4a). To account for variability of the intrablock structure in terms of geometry, six FEM models were built where the seed locations of each explicit set of vein elements were randomized. An example of the models showing the explicit structure and mesh elements is shown in Figure 6-4b. The mean geometry properties of the vein elements are listed in Table 6-1.
Figure 6-4: (a) Photo of drift heading in case study area with major veins traced in yellow; (b) example detail of composite FEM model from this study.

Table 6-1: Geometry properties of vein sets

<table>
<thead>
<tr>
<th>Properties</th>
<th>Units</th>
<th>Set 1</th>
<th>Set 2</th>
<th>Set 3</th>
<th>Set 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dip</td>
<td>Degrees</td>
<td>-77</td>
<td>5</td>
<td>-25</td>
<td>85</td>
</tr>
<tr>
<td>Length</td>
<td>Meters</td>
<td>20</td>
<td>10</td>
<td>10</td>
<td>1</td>
</tr>
<tr>
<td>Spacing</td>
<td>Meters</td>
<td>0.6</td>
<td>2</td>
<td>1</td>
<td>0.5</td>
</tr>
<tr>
<td>Persistence</td>
<td>N/A</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
<td>0.6</td>
</tr>
</tbody>
</table>

There are three sets of stiffness and strength properties for the quartz veins, which correspond to the proposed concept of primary, secondary, and tertiary states for intrablock structure. The strength behaviour is described by parameters from the Mohr-Coulomb criterion (Table 6-2). The normal and shear stiffness values, as well as the Mohr-Coulomb strength properties, for the primary state are based on previous results of numerical vein property calibration by the author (Chapter 5; Day et al., 2014). The primary residual state is a necessary segue to the secondary state in the modelling software used for this study. For the secondary state, the stiffness values are based on recommendations for clean, rough joints from the literature (e.g. Goodman et al., 1968 and Hungr and Coates, 1978), and the tertiary strength values are based on data of joints in quartzite from Read and Stacey (2009).
Table 6-2: Vein stiffness and Mohr-Coulomb strength properties

<table>
<thead>
<tr>
<th>Properties</th>
<th>Units</th>
<th>Primary peak</th>
<th>Primary residual</th>
<th>Secondary</th>
<th>Tertiary</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal stiffness, $K_n$</td>
<td>MPa/m</td>
<td>6,500,000</td>
<td>6,500,000</td>
<td>60,000</td>
<td>60,000</td>
</tr>
<tr>
<td>Shear stiffness, $K_s$</td>
<td>MPa/m</td>
<td>6,500,000</td>
<td>6,500,000</td>
<td>10,000</td>
<td>10,000</td>
</tr>
<tr>
<td>Tensile strength, $\sigma_t$</td>
<td>MPa</td>
<td>0.1</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Cohesion, $c$</td>
<td>MPa</td>
<td>4.3</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Friction angle, $\phi$</td>
<td>Degrees</td>
<td>25</td>
<td>40</td>
<td>40</td>
<td>25</td>
</tr>
</tbody>
</table>

6.6.2 Results and Analysis

In all FEM models of the case study drift, the secondary-tertiary states showed additional failure of both veins and material over the primary state. The maximum overbreak in the FEM models reached 1.8 m in the roof of model 1 (Figure 6-5).

![Figure 6-5: Model results of primary (first model) and secondary-tertiary (second model) states for three representative FEM models, with contours of maximum principal stress ($\sigma_1$); detail of model excavation geometry is shown at the bottom-right](image)

Note that only the three most representative FEM model realizations (mean, maximum, and minimum overbreak responses) of the six that were computed are shown in Figure 6-5. The extent of failure around the FEM models was selected around the interconnected yielded vein elements; solitary
yielded veins outside of these boundaries are assumed to have relatively little impact on overbreak development.

Representative cross-sections of the surveyed overbreak profiles along the case study drift are shown in Figure 6-6. There is a total of 69 overbreak profiles along the 340 m drift section. The relative frequency (in %) of overbreak from the profiles is divided into bins defined up to the maximum overbreak measurement in each example section (sections A-D in Figure 6-6). For example, section C shows that 6% of the profiles along the drift have a maximum roof overbreak from approximately 1.2 m and up to 1.8 m.

The maximum overbreak found in the FEM models was compared to that of the drift profile measurements from the case data. The range of overbreak results in the secondary-tertiary FEM models matches 96% of overbreak patterns in the case study drift, which fall within the bins represented by cross-sections A (25% frequency), B (65%) and C (6%) in Figure 6-6. While the maximum overbreak cases in the drift (represented by cross-section D in Figure 6-6) are not seen in the FEM models, this can be attributed to the anomalous (4%) frequency of these cases. In comparison, the overbreak results in the primary FEM models match approximately only 80% of the overbreak patterns in the case drift.

Figure 6-6: Representative cross-sections of case study drift showing overbreak around arched excavations, and relative frequency of occurrence along the 340 m section of drift, based on 69 profile measurements
6.7 Discussion and Conclusions

Discrete simulation of fractures has become a more commonplace model analysis technique for excavations in jointed rockmasses. Special attention is also required to simulate veins and other forms of intrablock structure within a model. In particular, the selection and evolution of stiffness and strength values for the model discontinuity elements must follow a different logic than that adopted for fractures and true joints. A new approach to better represent the behaviour of intrablock structure in discrete and explicit numerical models was proposed and tested herein using FEM analysis and case study data.

This approach changes stiffness and strength values for intrablock structure between pre-peak (primary), post-peak (secondary), and ultimate (tertiary) states. In the primary state when the intrablock structure is intact, the stiffness and strength properties are related to the intact stiffness of the infilling material. In the secondary state just after brittle failure, the stiffness and strength values should be closer to those of interblock structure with rough and clean surfaces, as features of the new fracture surface will control the subsequent behaviour of the intrablock structure in the rockmass. The tertiary state represents a residual condition of the fracture surface after some shear displacement.

The extent of overbreak in the case study drift is compared to FEM models of the drift with explicit intrablock structure. Overall, the FEM models in the secondary-tertiary stage match 96% of overbreak patterns along the case drift, while primary stage models match only 80%. The better match of secondary-tertiary stage models to the overbreak profile measurements indicates that the three-stage Mohr-Coulomb method proposed in this paper, introducing primary, secondary, and tertiary vein stiffness and strength states, is a good option to model the extent of overbreak along a drift in a tectonically complex rockmass with intrablock structure.

Further assessment of this methodology using geomechanical software with more flexible code customization is recommended to improve the threshold definition between primary, secondary, and tertiary stages using strength functions with strain dependent limits.
6.8 References


Chapter 7

Common Core: Core Logging Procedures for Characterization of Complex Rockmasses as Input into Geomechanical Analysis for Tunnel Design

7.1 Introduction

Rockmass characterization is an essential component of geotechnical design for tunnelling and modern underground mine infrastructure. Bid-stage design for tunnels relies heavily on and often exclusively on borehole characterization for data inputs. While traditional mining methods rely on assessing rock conditions within each blast round, modern block caving involves the development of hundreds of kilometers of tunnel infrastructure before any ore production begins, often through similar contracting practices to tunnels and their methods of drill core characterization. The design of deep geological repositories (DGRs) for the permanent storage of nuclear waste also requires several kilometers of excavation design and construction plans that are based on drill core characterization before construction.

The nature of data collected for conventional geotechnical characterization of drill core is often directed by inputs defined by empirical rockmass classification systems, such as Q (Barton et al., 1974) and Rock Mass Rating (RMR) (Bieniawski, 1976, 1989). At the time when these systems were introduced, no practical numerical tools for routine use were available, so design relied on an empirical process. Since then, numerical modelling has become a very powerful and ubiquitous design tool. Numerical methods have grown from their early elastic and homogeneous continuum behaviour to having complex elastoplastic and fully discrete capabilities. The advancement of modelling has allowed for more detailed analyses, including technically challenging underground excavations in more complex rockmasses, leading to significant improvements to design practice.

While significant improvements in the level of sophistication of numerical tools have been achieved, conventional core logging practices, whose procedural design predates numerical analysis, have

5 Parts of this chapter are published in a North American journal with the following citation: Day, J. J., Diederichs, M. S. and Hutchinson, D. J. 2015. Common Core: Core logging procedures for characterization of complex rockmasses as input into geomechanical analysis for tunnel design. Tunnels and Tunnelling. v2015:1:p26-32.
not made similar advancements. Conventional core logging practices do not capture the sophisticated data required for numerical input parameters, especially in complex rockmasses such as nodular argillaceous sedimentary rock or hydrothermally altered rock, where the geological features that exert significant controls on rockmass strength are ignored in common classification systems.

Hydrothermally altered rock is typically encountered in mining porphyry and other genetically similar ore deposits. In addition, Andean tunnels in South America for water, railway, and road transportation infrastructure are currently being driven through hydrothermally altered intrusive, volcanic and sedimentary rock. These young, altered rocks that are pervasive throughout the Andes make underground infrastructure development in this region unique because these rocks are a new frontier for tunnel designers.

Geological settings with complex rockmasses contain healed structures such as nodules and veins that exist within joint-bounded blocks of traditionally “intact” rock. These healed or partially healed structures (termed intrablock structure) within joint-bounded blocks (where the joints are the interblock structure) have a significant impact on rockmass strength. The author previously proposed methods to estimate the strength of a rockmass that contains both interblock and intrablock structure: at the structural scale using GSI (the Compsite GSI approach; Day et al. 2012a, 2012b), and at the intact drill core scale using intact strength parameters.

To advance rockmass characterization practices for complex rockmasses, several new core logging procedures have been developed and tested that account for healed structures present in such rockmasses. When compared to conventional methods, there are increasing amounts of detail in the recorded data for each method. Data from drill core sections is used to discuss the implications of each method regarding the level of detail, logging time, and cost. Two cases are used to illustrate the applicable core logging methods in the context of numerical models. The first is from a 20 m section of drill core in a hydrothermally altered andesitic tuff applied to the numerical design of a circular tunnel. The second case is from a 15 m section of drill core in nodular argillaceous sedimentary limestone for the design of an excavation for nuclear waste storage.
7.2 Conventional Core Logging

Rockmass data collected from drill core in conventional logging varies between users and projects, where not all available conventional data types are logged. Examples of rockmass data collected from core by three groups are compared in Table 7-1. There is a disconnect between data collected by the underground mine and shallow tunnel project, where data is collected just for the Q system, and a Canadian consulting company field procedures manual, where the standard is to collect data for the RMR system. It already becomes difficult to compare data from these three groups directly.

Table 7-1: Examples of rockmass data collected from core

<table>
<thead>
<tr>
<th>Data Collected</th>
<th>Underground Mine</th>
<th>Shallow Tunnel</th>
<th>Consulting Procedures Manual</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock unit description</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Total core recovery</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Solid core recovery</td>
<td></td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>RQD</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Fracture count</td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fracture spacing</td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Notable features (e.g. fault zone, disking, broken core)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Joint Set Number, (J_n)</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Joint Roughness Number, (J_r)</td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Joint Alteration Number, (J_a)</td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Discontinuity condition, (J_{con})</td>
<td></td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>Joint Roughness Coefficient</td>
<td></td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>Joint Compressive Strength, JCS, from Schmidt Hammer</td>
<td></td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>Infill type and thickness</td>
<td></td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>Point load strength index</td>
<td></td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>Veins</td>
<td></td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>Dip of structure with respect to core axis (Alpha angle)</td>
<td></td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>Structure dip direction angle about core axis (Beta angle)</td>
<td></td>
<td></td>
<td>X</td>
</tr>
</tbody>
</table>
7.3 Characterization Beyond Empirical Classification

The RMR and Q systems were designed to be empirical methods for tunnel support design. After decades of relying on these systems to assess rockmass quality, there was a need for a characterization system that would be dependent on direct geological field observations. The Geological Strength Index (GSI) emerged in 1995 in response to this demand and was designed to apply to all rock types normally encountered underground (Hoek et al., 1995). It has since developed into a standard tool for rockmass characterization at the outcrop and tunnel scale.

GSI has become a common interface for geological rockmass data from outcrop and tunnel mapping (but not directly from core logging) to numerical model inputs using the Hoek-Brown strength criterion (Hoek et al., 2002). The use of field observation as a direct input to numerical material properties has been very useful for the development of homogenous continuum models, and to some extent, heterogeneous models with discretized or explicit rockmass structure.

7.4 Including Intrablock Structure in Rockmass Characterization

The method developed for GSI in complex rockmasses in Day et al. (2012b) (the Composite GSI approach) is summarized here, as applied to the linearized and quantified GSI chart by Hoek et al. (2013). The conventional use of the GSI system dictates that when evaluating a rockmass that contains interblock and intrablock structure, the worst cases from each block size and joint condition ranking should be combined to give an overall GSI value for the rockmass. Using the hypothetical example shown in Figure 7-1, the combination of the interblock structure (square) and intrablock structure (diamond) would give a conventional worst case GSI value of 39 (white circle). This method has been shown in field observations to underestimate the rockmass strength. The Composite GSI (GSI* or CGSI) method proposed in Chapter 3 of this thesis calculates a more realistic GSI value for a rockmass that contains both interblock and intrablock structure, of 54 as shown by the green circle in Figure 7-1. The effect of GSI values on the Hoek-Brown strength envelopes for the hypothetical cases shown on the GSI chart in Figure 7-1 is shown in Figure 7-2, as well as the equations for the Hoek-Brown strength criterion.
Figure 7-1: The Geological Strength Index (GSI) chart, modified after Hoek et al. (2013), showing estimates of the GSI ratings for the interblock (square), intrablock (diamond), conventional GSI approach (white filled circle and orthogonal lines), and new composite GSI approach (filled circle and curved line)
In the updated GSI chart by Hoek et al. (2013), GSI values can be determined quantitatively by summing the two linear scales representing the discontinuity surface conditions (scale A) and the interlocking of rock blocks defined by these intersecting discontinuities (scale B) (see Figure 7-1).

\[ GSI = A + B \]  \hspace{1cm} (7.1)

According to Hoek et al. (2013), scale A can be estimated using the 4th Factor for Joint Condition (JCond\textsubscript{76}, or JCond\textsubscript{89}) in Bieniawski’s RMR classification (Bieniawski 1976; 1989) or Joint Roughness (J\textsubscript{r}) and Joint Alteration (J\textsubscript{a}) from Barton’s Q classification (Barton et al. 1974), as shown in Equation 7.2. Likewise, scale B can be estimated using the Rock Quality Designation (RQD) by Deere (1963) as in Equation 7.3. Scale B can also be estimated using the quantified GSI chart by Cai et al. (2004), as shown in Equation 3.4.

\[ A \approx 1.5(JCond\textsubscript{89}) \approx 1.8(JCond\textsubscript{76}) \approx 52(J_r/J_a)(1+J_r/J_a) \]  \hspace{1cm} (7.2)

\[ B \approx \frac{RQD}{2} \]  \hspace{1cm} (7.3)

\[ B \approx 20/3 \times \log_{10} (\text{Block Vol. in cm}^3) = 20 \times \log_{10} (\text{Equiv. Orthogonal Spacing in cm}) \]  \hspace{1cm} (7.4)

The new Composite GSI method for intrablock structure is described below with Equations 7.5 to 7.7. For a rockmass containing multiple, distinct suites of structure, an effective block size B* can be calculated from the individual spacings for each discontinuity set, while a weighted composite value for A* can be obtained considering the relative contribution of each structural set to rockmass integrity. In Equations 7.2 and 7.3, A\textsubscript{1}, B\textsubscript{1} apply to the first system (e.g. a clean, rough set of interblock structure), A\textsubscript{2}, B\textsubscript{2} apply to the second system (e.g. an infilled, smooth set of interblock structure), and A\textsubscript{n}, B\textsubscript{n} apply to the nth system (e.g. intrablock structure). This method becomes a powerful tool for rockmasses that contain multiple, distinct systems of structure because each structure can be considered individually before being combined into a single value that describes the entire rockmass (Day et al. 2013a). In this scenario, any number of structures (e.g. third, fourth, nth) can be included in the calculations. For Composite GSI (GSI* or CGSI):

...
\[
A^* = \left( \frac{A_1}{B_1} + \frac{A_2}{B_2} + \ldots + \frac{A_n}{B_n} \right)
\]
\[
B^* = 20 \log_{10} \left( \left( 10^{-B_1/20} + 10^{-B_2/20} + \ldots + 10^{-B_n/20} \right)^{-1} \right)
\]
\[
CGSI = GSI^* = A^* + B^*
\]

Figure 7-2: Generalized Hoek-Brown (Hoek et al. 2002) strength envelopes reflecting the various GSI ratings for the hypothetical example discussed in Figure 7-1. Equations that define the Hoek-Brown criterion are included in the figure to show the role of GSI

7.4.1 Incorporating Intrablock Structure into Laboratory Testing
Intrablock structure is typically ignored by geotechnical loggers because they are trained to assess fractures only. Moreover, to follow conventional strength testing guidelines, intact samples containing intrablock structure are deliberately excluded as much as possible from laboratory strength testing. In a
rockmass with pervasive healed structure, selecting the few intact samples without intrablock structure is not representative of the rockmass. Instead, intrablock structure should be included in geotechnical core logging, and a variety of intact samples with and without it should be selected for laboratory testing to improve the assessment of the rockmass behaviour. Codelco-Chile El Teniente Division has addressed this issue by developing a methodology to include intrablock structure in laboratory testing (Marambio et al. 1999), shown in Figure 7-3. Here, intrablock structure is recognized as a major contributor to rock strength and geotechnical data for the different failure modes provides a more comprehensive understanding of rock properties for numerical design.

![Figure 7-3: Classification of rock sample failure mode during testing (modified after Marambio et al. (1999))](image)
7.5 Incorporating Intrablock Structure into Core Logging

Four core logging procedures with increasing levels of data capture were developed to compare the effectiveness of the data collected in each for geotechnical design decisions. The methods were developed and tested using drill core in a hydrothermally altered porphyry deposit in northern Chile, as well as exploration boreholes in a magmatic Ni-Cu sulphide deposit at an active mine in Sudbury, Canada. Data for each logging method was collected from drill core sections each measuring 20 m long.

The four logging methods and sub-methods are summarized as follows and the components are described in detail in Table 7-2.

- Method 1: Traditional classification for empirical design (Q and RMR systems), for basic numerical models ((1a) unoriented and (1b) oriented core);
- Method 2: (2a) State of practice geotechnical logging (based on the method used by GeoBlast in Chile), plus (2b) basic information about intrablock structure (veins in unoriented core only);
- Method 3: Method 2a for interblock structure plus moderate detail of intrablock structure (veins in unoriented core only);
- Method 4: Method 2a for interblock structure plus high detail of intrablock structure for extremely detailed numerical models ((4a) unoriented and (4b) oriented core).

The block size measurement of intrablock structure is an assessment of spacing between veins. Only veins that are visible around the full circumference of the drill core are included. The measurement is systematically taken from the top inflection point of the vein around the outside of the drill core to avoid double counting veins (see Figure 7-4). The type of intrablock structure recorded refers to the mineralogical composition. The types of veins are classified by strength (“Strength Class”) based on a qualitative assessment of mineralogy and the quality of the bond between the vein and the wall rock. Recording the intrablock strength class becomes important for vein mineralogies that can be rockmass strengthening (e.g. quartz), but only if the bond to the wall rock is very good and the vein and wall rock appear fused or welded together. The thickness, alteration halo (if present), and Mohs’ hardness rating each provide additional information about the intrablock structure.
Figure 7-4: Example measurement of vein block size. Veins are measured from the top inflection point, and only veins that are visible around the full circumference of the drill core are included.

The logging methods from 1 to 4 consider increasing amounts of data but take increasing amounts of time to complete (see Table 7-3). Measuring orientation data (as in methods 1b and 4b) adds a significant amount of time to the logging process. The relative data collected and the time required to log are significant decision-making factors regarding schedule and budget for any project, and it is important to understand the added benefit (if any) of each piece of data that is included in the core logging procedure to the subsequent design process.

To put the importance of logging time into context, two examples of current industry core logging times and cost that are quoted for contracts are as follows. A North American consulting company quotes $120-160 USD per hour for core logging projects with an estimated rate of 40 m per 10-hour day (~$30-40 USD/m) for oriented core and 100 m per 10-hour day (~$12-16 USD/m) for unoriented core.
Table 7-2: Detailed parameters of four tested core logging methods including new methods 3 and 4

<table>
<thead>
<tr>
<th>Data Collected</th>
<th>Method 1</th>
<th>Method 2</th>
<th>Method 3</th>
<th>Method 4</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Drill Run Data</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>From (m), To (m)</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Core recovery</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td><strong>Rockmass Data</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strength estimate (R0-R6)</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Subordinate strength estimate</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>RQD</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Fracture spacing (m)</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Number of fractures</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Length of pieces</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Number of pieces</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Total length</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td><strong>Structural Data</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Number of joint sets</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Joint roughness (Jr)</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Joint alteration (Ja)</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Joint condition (J/cond)</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Principal roughness</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Secondary roughness</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Joint Roughness Coeff. (JRC)</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Principal alteration</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Secondary alteration</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Principal infilling: type, thickness (mm), quality</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Secondary infilling: type, thickness (mm), quality</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td><strong>Rock Data</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lithology</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Mineralization</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Alteration</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Phase</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Geotechnical unit</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td><strong>Intrablock Structure</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Block size mode (cm)</td>
<td>2b</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Type</td>
<td>2b</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Strength class (1, 2, 3)</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Thickness (mm): max, min, mode</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Alteration halo (type)</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Mohs hardness number</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td><strong>Orientation Data</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Depth (m)</td>
<td>1b*</td>
<td>4b**</td>
<td>1b*</td>
<td>4b**</td>
</tr>
<tr>
<td>Alpha (degrees)</td>
<td>1b*</td>
<td>4b**</td>
<td>1b*</td>
<td>4b**</td>
</tr>
<tr>
<td>Beta (degrees)</td>
<td>1b*</td>
<td>4b**</td>
<td>1b*</td>
<td>4b**</td>
</tr>
</tbody>
</table>

*For interblock structure, **For interblock & intrablock structure
Table 7-3: Relative logging times (normalized to method 1)

<table>
<thead>
<tr>
<th>Core Logging Method</th>
<th>Unoriented</th>
<th>Oriented</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Conventional: Q+RMR</td>
<td>1</td>
<td>3.2</td>
</tr>
<tr>
<td>2. State of practice</td>
<td>1.2</td>
<td>3.5</td>
</tr>
<tr>
<td>3. Method 2+ basic veins</td>
<td>1.9</td>
<td>4.2</td>
</tr>
<tr>
<td>4. Method 2+ detailed veins</td>
<td>2.1</td>
<td>11.9</td>
</tr>
</tbody>
</table>

A typical Chilean consulting company quotes $15 to $65 USD per meter, depending on contracts, with an estimated rate of 30-50 m per day (mix of oriented and unoriented core). Based on these estimates, costs of core logging can range from $12,000 to $65,000 USD/km. If the logging rate decreases due to collection of more detailed data, the core logging costs could easily exceed $100,000 USD/km. These estimates are in 2013 dollars.

7.6 Implications of Data Input for Numerical Models

To test the design implications of the different core logging methods, two cases of drill core and excavation examples were conducted with numerical analyses. The first case is a circular tunnel in a partially leached andesitic tuff from Chile, and the second case is a tunnel from a nuclear waste deep geological repository in nodular argillaceous sedimentary limestone from Ontario, Canada. The excavation cases were simulated using the FEM software, Phase\(^2\) (RocScience, 2013b). While the intact rock properties and in situ stresses remain constant in the numerical analyses, the types, geometries, and strength properties of the rockmass structure vary. The resulting rockmass behaviours for both cases are analyzed and discussed.

7.6.1 Case 1: Hydrothermal Andesitic Tuff

For the first case, the input data was based on a 20 m drill core section of a partially leached andesitic tuff that was logged in Chile (Figure 7-5). Numerical models were generated to assess a circular 4 m-diameter tunnel designed to house a conveyor system for ore transportation in an operating mine. The tunnel was tested in five different rockmasses based on input data from core logging methods 1a, 2a, 2b, 3, and 4b discussed above. Method 1b was not included in this study as the focus is largely on comparisons of intrablock structure, and 4a was not included as 4b displaces it.
Figure 7-5: 20 m drill core section of a partially leached andesitic tuff that was logged using the discussed core logging methods; data collected from this core was used for the conveyor tunnel numerical analysis

213
7.6.1.1 Model Setup

The simulated conveyor tunnel example is approximately 500 m deep and in an in-situ stress state with a K ratio of 1.5:1 (H:V), such that $\sigma_1 = 21 \text{ MPa}$ (horizontal and perpendicular to the tunnel axis), $\sigma_2 = 17.5 \text{ MPa}$ (horizontal parallel to the tunnel axis), and $\sigma_3 = 14 \text{ MPa}$ (vertical). The intact lab strength properties for the rock material represent a partially leached andesitic tuff (see Table 7-4). Residual intact rock properties were based on a GSI of 65. The peak $\sigma_{ci}$ and $m_i$ values were selected based on a linear regression fit from tensile, Unconfined Compressive Strength (UCS), and triaxial laboratory test data, as shown in Figure 7-6.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s Modulus (MPa)</td>
<td>38,250</td>
</tr>
<tr>
<td>Poisson’s Ratio</td>
<td>0.23</td>
</tr>
<tr>
<td>Intact compressive rock strength, $\sigma_{ci}$ (MPa)</td>
<td>44</td>
</tr>
<tr>
<td>$m_i$</td>
<td>11</td>
</tr>
<tr>
<td>GSI</td>
<td>100</td>
</tr>
</tbody>
</table>

Table 7-4: Lab strength properties of the simulated partially leached andesitic tuff

A schematic of the general model geometry and boundary conditions for all models is shown in Figure 7-7. The geometry and properties of structures in the discrete fracture network, and the corresponding GSI value in the homogenous equivalent continuum, vary between models. The continuum section of the model is included to lessen the computation time required for the models and was calculated using the Composite GSI approach for the applicable suites of interblock and intrablock structure. The relative distances between the tunnel, discrete zone, and continuum zone in Figure 7-7 are approximately to scale.
Figure 7-6: Strength envelopes of intact rock (H-B) and three vein types (M-C) used in models, including both peak and residual properties. Peak intact strength envelope is a regression fit from tensile, UCS and triaxial test data.

Figure 7-7: Schematic of the general model geometry used for all models.
7.6.1.2 Joint Properties

The model based on logging method 1 only considers joints. The Mohr-Coulomb failure criterion was selected, where the tensile strength is zero, cohesion (c) was assumed to be near zero (0.001 MPa for numerical stability) and the friction angle (ϕ) was determined using the following relationship of Q parameters:

\[ \phi = \tan^{-1}\left(\frac{J_r}{J_a}\right) \]  

(7.8)

where \( J_r \) represents joint roughness and \( J_a \) represents joint alteration (Barton & Bandis, 1990). Like method 1a, the model based on logging method 2a considers only joints. However, Barton-Bandis shear strength parameters are collected, leading to the use of the Barton-Bandis joint failure criterion for the corresponding tunnel model. Joint strength properties are shown in Table 7-5.

**Table 7-5: Joint strength properties in andesitic tuff**

<table>
<thead>
<tr>
<th>Model No.</th>
<th>1</th>
<th>2a, 2b, 3, 4b</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fracture spacing (cm)</td>
<td>48.5</td>
<td>48.5</td>
</tr>
<tr>
<td>No. of sets</td>
<td>2.5</td>
<td>2.5</td>
</tr>
<tr>
<td>Normal stiffness (MPa/m)</td>
<td>19000</td>
<td>19000</td>
</tr>
<tr>
<td>Shear stiffness (MPa/m)</td>
<td>1900</td>
<td>1900</td>
</tr>
<tr>
<td>Joint roughness factor, ( J_r )</td>
<td>2.1</td>
<td>-</td>
</tr>
<tr>
<td>Joint alteration factor, ( J_a )</td>
<td>2.2</td>
<td>-</td>
</tr>
<tr>
<td>Peak friction angle (°)</td>
<td>43</td>
<td>-</td>
</tr>
<tr>
<td>Cohesion (MPa)</td>
<td>0.001</td>
<td>-</td>
</tr>
<tr>
<td>Residual friction angle (°)</td>
<td>21*</td>
<td>31**</td>
</tr>
<tr>
<td>Residual cohesion (MPa)</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>Joint roughness coefficient, JRC</td>
<td>-</td>
<td>9.5</td>
</tr>
<tr>
<td>Joint wall compressive strength, JCS (MPa)</td>
<td>-</td>
<td>22</td>
</tr>
</tbody>
</table>

*assume 50%, **after Read and Stacey (2009)

JCS was based on a “fair” GSI joint wall condition, corresponding to 55% of the intact rock compressive strength (Pitts and Diederichs, 2011). Joint stiffness properties were not tested in the field as part of the core logging campaign. As such, the joint stiffness properties were based on data in Read and Stacey (2009). Since method 2a of core logging is regarded as a more sophisticated approach, the joint geometry and properties used in that model were used in the models for the subsequent core logging methods for consistency.
7.6.1.3 Vein Properties

The models based on logging methods 2b, 3, and 4b contain both joints and veins. Vein strengths were assessed qualitatively in the field and grouped based on relative strength due to mineralogy and observed competence in the drill core. Quantification of these strength observations was guided by previous vein strength calibration (e.g. Day et al., 2013b) and are based on a brittle fracture model (Diederichs, 2007). The strengths used in these models are shown in Table 7-6. The veins were modelled explicitly using Voronoi joint networks, which create polygonal shapes with high angularity and in an irregular pattern, and are widely accepted for modelling microstructures in rock (e.g. Lan et al., 2010). The vein geometries change between models due to the different information collected between logging methods 2b to 4 (see Table 7-7).

**Table 7-6: Vein strength properties in andesitic tuff**

<table>
<thead>
<tr>
<th>Vein type</th>
<th>Strong</th>
<th>Average</th>
<th>Weak</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal stiffness (MPa/m)</td>
<td>80000</td>
<td>60000</td>
<td>40000</td>
</tr>
<tr>
<td>Shear stiffness (MPa/m)</td>
<td>36000</td>
<td>27000</td>
<td>18000</td>
</tr>
<tr>
<td>Peak tensile strength (MPa)</td>
<td>0.1</td>
<td>0.08</td>
<td>0.05</td>
</tr>
<tr>
<td>Peak friction angle (°)</td>
<td>20</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>Peak cohesion (MPa)</td>
<td>4</td>
<td>3</td>
<td>2</td>
</tr>
<tr>
<td>Resid. tensile strength (MPa)</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Resid. friction angle (°)</td>
<td>30</td>
<td>23</td>
<td>15</td>
</tr>
<tr>
<td>Resid. cohesion (MPa)</td>
<td>0.1</td>
<td>0.08</td>
<td>0.05</td>
</tr>
</tbody>
</table>

**Table 7-7: Vein geometry in andesitic tuff**

<table>
<thead>
<tr>
<th>Vein set</th>
<th>Strong</th>
<th>Avg.</th>
<th>Weak</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model 2b. Block size (cm)</td>
<td>-</td>
<td>7</td>
<td>-</td>
</tr>
<tr>
<td>Model 3. Block size (cm)</td>
<td>20</td>
<td>-</td>
<td>12</td>
</tr>
<tr>
<td>Model 4b. Block size (cm)</td>
<td>10</td>
<td>-</td>
<td>19</td>
</tr>
<tr>
<td>Model 4b. Apparent dip (°)</td>
<td>62</td>
<td>-</td>
<td>24</td>
</tr>
</tbody>
</table>

In method 2b, the measured block size includes all vein types; therefore, an average block size and strength of all veins contained in the rockmass are used in the model. In method 3, veins and their block sizes are separated by strength class (e.g. rockmass weakening but strong, and rockmass weakening but
weak), so vein types can be modelled using separate networks. In method 4b, additional orientation information is collected where the strike and dip of each vein are measured. In the drill core used in this study, two sets of vein orientations were identified using stereonet analysis (see Figure 7-8 for stereonet and Table 7-7 for apparent dips). Although it was possible to separate the vein orientations by strength class (which results here in a total of 4 vein sets), only two sets were modelled due to computational limitations. Therefore, the strong vein strength properties and weak vein strength properties (as in Table 7-6) were assigned to each of the two modelled vein sets. In order to model anisotropic veins in Phase³ to represent the identified sets, manual scaling, and rotation of the Voronoi networks was required for each vein set. A detailed view of the anisotropic Voronoi vein networks is shown in Figure 7-9.

![Stereonet analysis of vein orientations in Dips software by RocScience (2013a), where the strike/dip of the mean set planes are oriented at 332/57° and 047/74°](image)

Figure 7-8: Stereonet analysis of vein orientations in Dips software by RocScience (2013a), where the strike/dip of the mean set planes are oriented at 332/57° and 047/74°
7.6.1.4 FEM Numerical Model Results

Several observations can be made from the models representing each of core logging methods 1a, 2a, 2b, 3, and 4b. An overview of the major principal stresses ($\sigma_1$) in the models is shown in Figure 7-10. Detailed views of the model results, in terms of $\sigma_1$, total displacement, and yielded mesh elements are shown in Figure 7-11.

A comparison of joint properties can be made between models 1 and 2a. In the respective core logging methods, joint properties are recorded using Q parameters in method 1 (which can be approximated to use of the Mohr-Coulomb failure criterion for joint strength) and Barton-Bandis shear strength criterion parameters in method 2a. From 1 to 2a, there is a reduction in the extent of both yielded joint elements and yielded mesh elements away from the excavation. In addition to surface support on the excavation wall, the depth of yield from the excavation becomes important for penetrative support design (e.g. length of rockbolts). This suggests that the simplest approximation of joint strength in method 1 is an underestimate.
There is a significant increase in total displacement around the tunnel when veins are included in the models. As the detail of veins increases from model 2b to 4b, the total displacement at the excavation boundary increases, as well as the depth at which the rockmass is affected. It is also interesting that ultimately the outer limit of total displacement is controlled by joints (most pronounced in model 4b). Rockbolt design for this tunnel example based on models 1, 2a, or 2b may underestimate the length required for effective support, if the real rockmass behaviour is closer to that in model 4b. While the depth of joint yield from the tunnel boundary is approximately equal in all directions, the vein yield is more sensitive to the in-situ stress K ratio of 1.5:1 (H:V), where the majority of vein yield occurs above and below the excavation.

Many of the yielded mesh elements in models 1 and 2a are present as yielded vein elements in models 2b, 3 and 4b. The yielded mesh elements in 4b tend to align parallel to an average of the two vein network apparent dips even though most vein failure occurs in the weaker set. This may be due to the general influence on induced stress from both vein sets as opposed to the yielded veins themselves.

The anisotropy of the vein networks in model 4b has an effect on the medium-field stresses where there is a pinwheel effect visible on the stress contours (see Figure 7-10).
Figure 7-10: Overview of models showing progressive detail and major principal stress ($\sigma_1$) contours. (Note 'Pinwheel' effect in model 4b due to vein network anisotropy)
Figure 7-11: Major principal stress ($\sigma_1$), total displacement, and yielded element results for all conveyor tunnel models in hydrothermally altered andesite
Model 1 does not capture the same rockmass behaviour as model 4b. Comparisons of yielded joints and depth from the excavation boundary of yielded material for each model are shown in Table 7-8. The greater detail collected in method 4b core logs suggests that model 4b is the most geologically accurate; however, the similarities between models 3 and 4b in Table 7-8 suggest that method 3 may be accurate enough to compromise between logging time and numerical results.

**Table 7-8: Quantitative numerical results of tunnel in andesitic tuff**

<table>
<thead>
<tr>
<th>Method</th>
<th>1</th>
<th>2a</th>
<th>2b</th>
<th>3</th>
<th>4b</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yielded joint elements</td>
<td>779</td>
<td>571</td>
<td>14986</td>
<td>44374</td>
<td>12660</td>
</tr>
<tr>
<td>Total joints</td>
<td>2642</td>
<td>2642</td>
<td>78178</td>
<td>121546</td>
<td>36416</td>
</tr>
<tr>
<td>% yielded joints</td>
<td>29%</td>
<td>22%</td>
<td>19%</td>
<td>37%</td>
<td>35%</td>
</tr>
<tr>
<td>Avg. depth of yield (m)</td>
<td>0.75</td>
<td>0.67</td>
<td>0.59</td>
<td>0.69</td>
<td>0.65</td>
</tr>
</tbody>
</table>

Overall, these models show that different levels of structural detail, associated with different levels of core logging resolution in terms of data depth and breadth, produce significant variations in predicted tunnel response with respect to total displacements and yielded mesh and joint elements.

**7.6.2 Case 2: Nodular Argillaceous Cobourg Limestone**

For the second case, the core logging methods were applied to a 15 m drill core section of nodular argillaceous sedimentary limestone (see Figure 7-12). Numerical simulation of a proposed tunnel section in an underground deep geological repository (DGR) for high-level nuclear waste (HLW) storage was conducted. The arched tunnel design also has a vertical hole in the floor for nuclear waste canister storage. The excavation was modelled in two different rockmasses defined by input data from the applicable core logging methods, 1 and 3, discussed above.
Figure 7-12: 15 m drill core section of a nodular limestone that was logged using the discussed core logging methods. Data collected from this core was used for the numerical analysis of the DGR.
7.6.2.1 Model Setup

This idealized repository excavation example was assumed to be approximately 500 m deep and in an in-situ stress state with ratios of 2:1 (σ_H:σ_v) and 1.6:1 (σ_H:σ_v). The intact strength properties for the rock material represent a nodular argillaceous limestone and are based on published values (NWMO, 2011) as shown in Table 7-9. The peak strength values were selected based on a linear regression fit from tensile, Unconfined Compressive Strength (UCS), and triaxial laboratory test data, as shown in Figure 7-13. Rockmass parameters are specified using the GSI and Hoek-Brown (H-B) approach (Hoek et al., 2002). Alternatively, the strength envelopes can be simplified to equivalent linear Mohr-Coulomb (M-C) envelopes.

Table 7-9: Intact rock properties of argillaceous nodular limestone

<table>
<thead>
<tr>
<th>Generalized Hoek-Brown Parameters</th>
<th>Peak</th>
<th>Residual</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit weight (MN/m³)</td>
<td>0.026</td>
<td>N/A</td>
</tr>
<tr>
<td>Intact Young’s Modulus (GPa)</td>
<td>40</td>
<td>N/A</td>
</tr>
<tr>
<td>Poisson’s Ratio</td>
<td>0.3</td>
<td>N/A</td>
</tr>
<tr>
<td>Unconfined Compressive Strength, σ_u (MPa)</td>
<td>113</td>
<td>N/A</td>
</tr>
<tr>
<td>m_i / m_b</td>
<td>17</td>
<td>2.85</td>
</tr>
<tr>
<td>s</td>
<td>1</td>
<td>0.0039</td>
</tr>
<tr>
<td>a</td>
<td>0.4</td>
<td>0.506</td>
</tr>
<tr>
<td>GSI</td>
<td>100</td>
<td>50</td>
</tr>
</tbody>
</table>

A schematic of the general model geometry and boundary conditions for both models is shown in Figure 7-14. The geometry and properties of structures in the discrete fracture network, and the corresponding GSI value in the homogenous equivalent continuum, vary between models. The continuum section of the model is included to lessen the computation time required for the models and was calculated using the Composite GSI approach for the applicable suites of interblock and intrablock structure.
Figure 7-13: Peak and residual strength envelopes for intact rock (H-B) and intrablock structure (H-B converted to M-C) used in models. Peak intact strength envelope is a regression fit from tensile, UCS and triaxial test data, and peak intrablock strength is the minimum boundary of the laboratory test data.
7.6.2.2 Joint and Vein Properties

The model based on logging method 1 only considers interblock structure: joints and bedding. The selection of stiffness and strength properties was aided by information in Read and Stacey (2009) and Pitts and Diederichs (2011), and the values are shown in Table 7-10. The model based on method 3 contains both interblock and intrablock structure. The intrablock stiffness properties were selected with help from Goodman (1968), and strength properties of the intrablock structure were selected as the lower bound regression fit of laboratory testing data, as shown in Figure 7-13. As in the first case, this intrablock structure was modelled explicitly using Voronoi joint networks.
### Table 7-10: Interblock and intrablock properties (peak / residual)

<table>
<thead>
<tr>
<th>Structure type</th>
<th>Subvertical joints</th>
<th>Bedding</th>
<th>Intrablock nodules</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average spacing (m)</td>
<td>13</td>
<td>1.5</td>
<td>0.1</td>
</tr>
<tr>
<td>Normal stiffness (MPa/m)</td>
<td>16,000</td>
<td>35,000</td>
<td>40,000</td>
</tr>
<tr>
<td>Shear stiffness (MPa/m)</td>
<td>12,000</td>
<td>10,000</td>
<td>20,000</td>
</tr>
<tr>
<td>Tensile strength (MPa)</td>
<td>-</td>
<td>0.01 / 0</td>
<td>3.9 / 0</td>
</tr>
<tr>
<td>Cohesion (MPa)</td>
<td>-</td>
<td>0.05 / 0</td>
<td>13 / 0.5</td>
</tr>
<tr>
<td>Friction angle (°)</td>
<td>-</td>
<td>26 / 24</td>
<td>41 / 32</td>
</tr>
<tr>
<td>Joint Wall Compressive Strength, JCS</td>
<td>55</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Joint Roughness Coefficient, JRC</td>
<td>10</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Residual friction angle, ( \phi_r )</td>
<td>31</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

#### 7.6.2.3 FEM Numerical Model Results

Several observations can be made from the models representing each of core logging methods 1 and 3. Detailed views of the model results, in terms of the maximum principal stress \( (\sigma_1) \), total displacement, and yielded mesh elements are shown in Figure 7-15. There is a significant increase in total displacement around the excavation when intrablock structure is included in the models since the weaker planes of intrablock structure allow the rockmass to accommodate more displacement. Rockbolt design for this excavation example based on the conventional logging approach, method 1, may underestimate the length required for rockbolts to be effective if the real rockmass behaviour is dependent on the influence of intrablock structure.
7.7 Design Implications and Conclusions

Four core logging procedures with increasing levels of data capture, including two new methods, were tested to compare the effectiveness of the data collected in each for geotechnical design decisions. The methods were developed and tested using drill core in a hydrothermally altered porphyry deposit in northern Chile, as well as exploration boreholes in a magmatic Ni-Cu sulphide deposit at an active mine in Sudbury, Canada. Two cases are used to illustrate the applicable core logging methods in the context of numerical models. The first is from a 20 m section of drill core in a hydrothermally altered andesitic tuff applied to numerical design of a circular tunnel. The second case is from a 15 m section of drill core in nodular argillaceous sedimentary limestone for the design of a nuclear waste storage excavation.

The numerical investigations using case examples of a circular conveyor tunnel and a deep underground repository excavation show that the input data based on different core logging methods has a significant influence on the results of rockmass behaviour. Conventional logging methods which consider
only interblock structure were compared to several methods developed and tested by the author that consider both interblock and intrablock structure.

The detail of these logging methods, from traditional rockmass classification parameters to current state of practice conventional logging and detailed intrablock data in oriented core, increases significantly, but the time required for logging also increases. While the models discussed in this study show that greater detail in logging results in more geologically accurate numerical models, there are consequences for the cost and schedule of a project.

Overall, the example case models show that minimal data input from conventional core logging results in the strongest apparent rockmass with the least amount of rockmass yield and displacements. Conversely, the models with more data input from the proposed detailed core logging procedures for complex rockmasses reveal the weakest apparent rockmass with the greatest amount of rockmass yield and displacements. The difference in rockmass behaviour between models for the circular tunnel in the hydrothermally altered rockmass with input from core logging methods 3 and 4 is minimal. This suggests that the investment in core logging time and finances is useful up to method 3, but the additional benefit of extreme detail captured in method 4 at a consequence of extra cost may not be necessary for all geotechnical projects.
7.8 References


Hoek, E., Carter, T., and Diederichs, M. S., 2013. Quantification of the Geological Strength Index Chart. 
San Francisco, CA, USA, 47th U.S. Rock Mechanics Geomechanics Symposium, ARMA.

Rotterdam, Balkema.


CODELCO-Chile El Teniente Division [in Spanish].


Pitts, M. and Diederichs, M.S. 2011. The effect of joint condition and block volume on GSI and rockmass 
strength estimation. 14th Pan-Am. Conf. on Soil Mech. and Geotech. Eng., CGS, Toronto, 
Canada.


Chapter 8

Mineralogical Assessment and Laboratory Direct Shear Testing of Intrablock Structure in the Cobourg Limestone

8.1 Introduction

Conventional characterization and analysis methods for geotechnical assessment in mining, civil tunnelling, and other excavations consider only the intact rock properties and the discrete fractures that are present and form blocks within rockmasses. Field logging and classification protocols are based on historically useful but highly simplified design techniques, including direct empirical design and empirical strength assessment for simplified ground reaction and support analysis. Given the comparatively complex, sophisticated and powerful simulation and analysis techniques now practically available to the geotechnical engineer, this research is driven by the need for enhanced characterization of joints and other fractures (interblock structure). Furthermore, as modern underground excavations go deeper and enter into more high stress environments with complex excavation geometries and associated stress paths, healed structures within initially intact rock blocks such as sedimentary nodules and hydrothermal veins, veinlets and stockwork (termed intrablock structure) are having an increasing influence on rockmass behaviour and should be included in modern geotechnical design. Due to the reliance on geotechnical classification methods which predate computer aided analysis, these complexities are ignored in conventional design. Intrablock structure governs stress-driven behaviour at depth, gravity driven disintegration for large shallow spans, and controls ultimate fragmentation.

Previous chapters have discussed material parameters and modelling strategies largely for hydrothermal vein type intrablock structure. Similar challenges exist within heterogeneous sedimentary rocks such as the Cobourg limestone that is being considered for nuclear waste storage in Ontario. With

---

6 Parts of this chapter have been submitted to an invited international refereed journal with the following citation: Day, J. J., Diederichs, M. S. and Hutchinson, D. J. 2016. New direct shear testing protocols and analyses for fractures and healed structures. Submitted to Engineering Geology Special Issue on “Characterization of Fractures in Rock: from Theory to Practice” (ROCKFRAC) Eds. X. Zhuang, D. Tannant, and G. Ma.
modern computer simulation tools, it is possible to have a variety of simulation strategies to consider this intrablock complexity using explicit or discrete representations of rockmass structure, as shown in Figure 8-1. These models require more sophisticated input related to shear strength parameters as well as stiffness parameters for explicit or discontinuous rockmass structure.

Figure 8-1: Examples of strategies to include explicit sedimentary intrablock structure in a FEM model at the excavation scale; interblock bedding is represented in both models with horizontal persistent structure elements (orange); intrablock structure can be represented by (left) non-persistent horizontal parallel structure elements or (right) Voronoi element geometry

The Cobourg Formation is an Ordovician argillaceous and nodular limestone with core-scale intrablock structure defined by anisotropic and heterogeneous variations in lithology. The rock has distinctive light grey and calcite-rich nodules surrounded by dark grey and clay-rich boundaries. In some cases, fossils or fossil fragments are present in the inter-nodular layers. The formation of the characteristic
nodular structure is a result of both physical and chemical compaction effects on a bioturbated, shell-rich carbonate material with interbedded clays that would have been located in a shallow shelf to shoal environment (Brookfield and Brett, 1988). The fossils, including crinoids and brachiopods, would have been fragmented during physical compaction. The formation of nodules and inter-nodular clay-rich seams are a result of either pressure solution or physical compaction processes. Clays, pyrite and detrital silicates, organic matter, and dolomite variably occur in the inter-nodular seams (Choquette and James, 1987). These inter-nodular boundaries are considered to be intrablock structure for this rockmass (see Figure 8-2). This stratigraphic unit at 680 m depth below the Bruce site near Kincardine, Ontario, has been selected by the Canadian Nuclear Waste Management Organization (NWMO) as the host rock for the underground deep geological repository (DGR) to store low and intermediate level nuclear waste.

Figure 8-2: Drill core of Cobourg limestone from the Bowmanville quarry
A comprehensive mineralogical investigation has been conducted to determine the compositions of the Cobourg limestone from the Bowmanville quarry owned by St. Marys Cement near Bowmanville, Ontario, Canada, and the Bruce DGR site near Kincardine, Ontario, Canada. Powdered X-ray diffraction (XRD) and scanning electron microscope (SEM) techniques are implemented in concert to determine the mineralogy and elemental compositions. Evaluation of the geometries and crack propagation tendencies of intrablock structure in thin section are also evaluated using SEM analysis.

The physical laboratory servo-controlled direct shear test program in this study is used to define structural stiffness and strength properties for targeted shear surfaces in the Cobourg limestone from the Bowmanville quarry. Conventional joint shear strength properties are evaluated and joint stiffness is analyzed in both of the normal and shear components. The importance of joint stiffness was recognized more recently than shear strength, with the need to describe pre-yield stress-deformation responses for the introduction of Finite Element and Finite Difference numerical methods. Joint stiffness can be a challenging parameter to evaluate and the current use of it is inadequate for detailed numerical models in advanced geotechnical design.

An extensive literature search for direct shear test results and other observations of joint shear stiffness shows many discussions that centre on the scale dependency of shear stiffness when it is defined by the peak secant stiffness value of the stress-displacement data. However, initial findings show that this scale dependency is reduced when using a pre-yield tangent stiffness value. The scale dependency reduces even further when a pre-yield chord stiffness value is used, which is a more appropriate scenario for numerical modelling. A thorough review and reanalysis of joint stiffness data of interblock and intrablock structures leads this investigation in an effort to understand more effective stiffness and strength properties. The methodologies for strength and stiffness analyses for direct shear testing of joints is extended here to intrablock structure in order to develop streamlined direct shear test analyses across both interblock and intrablock types of rockmass structure. The reanalysis of stiffness and consistent approach across all structure aim to provide suitable laboratory testing results for inputs to geomechanical parameters in numerical models with explicit or discrete structure.
This direct shear test program investigates the effects of changing confining stresses and sample scales for tests through shear surfaces of interblock and intrablock structures. The normal stresses include 0.2, 0.5, 1.2, 2, 3, 5, and 8 MPa and the target shear sample sizes have diameters of 2 and 3 inches. The direct shear testing protocols have been developed at high quality standards that will ultimately be subject to public review with the engineering design of the DGR for NWMO.

Finally, a comparison is made between the direct shear results of this investigation on the Cobourg limestone from the Bowmanville quarry and direct shear test results reported by Canmet on Cobourg limestone samples from boreholes at the repository depth at the Bruce DGR site. The results of the mineralogy investigation will be used to explain the differences in the shear stiffness and strength results between the different sample source locations.

8.2 Motivation of Study
Deep Geological Repositories (DGRs) are intended to be a permanent storage solution for nuclear waste and have a design life of one million years. In order to effectively design the repository for such a large timeframe, state of the art numerical modelling tools are necessary to produce the best prediction of geomechanical and other ground behaviours. Therefore, a sound understanding of rockmass properties is required to effectively predict rockmass behaviour. The selection of mechanical properties of explicit rockmass structure elements has a significant influence on the model behaviour, so measuring these properties in the laboratory is therefore essential to improving model accuracy and behaviour prediction capabilities.

8.2.1 Deep Geological Repositories for Nuclear Waste Storage
The international use of nuclear fuel and other products since the mid-20th century has led to the accumulation of nuclear waste. This waste continues to be stored at ground surface and frequently in populated areas. The generally accepted long-term and permanent solution for the storage of nuclear waste in the international nuclear community is in DGRs. The DGR solution is generally designed to be sited in a rock unit that is deep enough below the ground surface to be isolated from the biosphere, on the
order of 500 m. These facilities depend on a multiple-barrier concept in addition to the construction depth, including the waste canister and bentonite backfill (see Figure 8-3). Canada is currently working toward the design and construction of DGRs for Low and Intermediate Level Waste and High Level Waste, and this initiative is operated by the Nuclear Waste Management Organization (NWMO) and Ontario Power Generation (OPG).

Figure 8-3: Illustration of a Canadian design for a Deep Geological Repository (Noronha, 2016)
The service design life for the DGRs is one million years in order to sufficiently isolate the waste radionuclide contaminants for their lifespan, which is by far the longest design life for any human engineered project. A prerequisite for the host rock where tunnels are constructed is stability in the geology, stress system, jointing, and faulting. Many potential host rocks have been identified, including crystalline basement, basalt, salt, tuff, and a range of argillaceous sedimentary units (McKinley, 2007). The sited DGR location for Canada’s Low and Intermediate Level repository is in a sedimentary rock sequence in southern Ontario, and sedimentary sequences are still under consideration for a High Level repository.

The sedimentary stratigraphic sequence in southern Ontario consists of carbonates, shales, sandstones, and evaporates that overlay the Precambrian basement rocks. The depth of excavation for the DGR at the Bruce site is approximately 680 m below surface within the Cobourg limestone formation, which is the youngest member of the middle Ordovician Trenton Group. The Cobourg limestone is capped by the 200 m thick upper Ordovician Queenston Formation and Georgian Bay Formation shales, and underlain by the limestones and shales of the middle Ordovician, Cambrian sandstone, and Precambrian felsic granitic gneiss. The shale cap above the Cobourg Formation has a very low permeability, which is an asset to the multiple-barrier concept that would slow the progression of radionuclides toward the surface and biosphere.

In southern Ontario, the Paleozoic stratigraphy is sub-horizontally bedded and essentially continuous. The dip of the stratigraphy shallows to the east so the Cobourg Formation occurs at quarry depth in Bowmanville, Ontario. The Bowmanville quarry presents the opportunity for extensive study of the Cobourg Formation and has been the source of rocks for several programs of laboratory testing for NWMO research, including this mineralogy and direct shear testing program.
Figure 8-4. Geological cross-section through the Michigan Basin with approximately 45x vertical exaggeration (modified after Gartner Lee Ltd., 2008a)

8.3 Source of Rock Samples

Canmet conducted direct shear tests on the DGR Cobourg from boreholes drilled at the Bruce site, but due to the limited availability of additional samples from the Bruce site for a comprehensive laboratory test program, the Cobourg limestone from the St. Mary Cement Bowmanville quarry near Bowmanville, Ontario, was used for this testing program as an analogue for the unit at depth. The Canmet data is reanalyzed as part of this investigation and compared to the results of tests on the Bowmanville quarry Cobourg limestone.

The Cobourg limestone occurs near ground surface at the Bowmanville quarry and shallowly dips NW toward the Bruce DGR site, where it is located at the selected repository depth of ~700 m below ground surface. The rock sourced from the Bowmanville quarry is an easily accessible analogue of the Cobourg limestone at the DGR site. It is therefore suitable for extensive geomechanical laboratory testing to characterize the rock with state-of-the-art research tools before DGR construction begins. The Cobourg limestone from the Bowmanville quarry is argillaceous and nodular, where light grey nodules of calcite-rich limestone are surrounded by tortuous layers of dark grey, clay-rich rock. The Cobourg limestone unit in southeastern Ontario is also called the Lindsay formation where it occurs closer to ground surface, as is the case at the Bowmanville quarry. These units are stratigraphically equivalent but may exhibit some
lateral changes on the kilometer scale, due to variations in depositional environments prior to lithification.

For the purposes of this research, the name Cobourg limestone is used. The Bowmanville quarry is located in UTM Zone 17T at 685516.05 Easting and 4861723.59 Northing coordinates (approximately 43°53'07"N, 78°41'26"W), and the specific source location of the laboratory samples from the quarry is shown in Figure 8-5 and Figure 8-6.

Figure 8-5: The St. Marys Cement Bowmanville quarry near Bowmanville, Ontario, Canada, is the source location of the Cobourg limestone for this direct shear laboratory testing program; a) geographical site context; b) aerial view of quarry with precise source location indicated at location d,e (image courtesy of Google); c) view of source location; d) quarry wall at source location; e) rock block pile is source of laboratory testing samples (photographs courtesy of M. Diederichs)
Figure 8-6: Cross-section of proposed overall final slope geometry at the St. Marys Cement Bowmanville quarry; source level of Cobourg samples used for direct shear testing program is shown (courtesy of St. Mary’s Cement)

8.3.1 Sample Drilling Procedure

Cylindrical core samples of the Cobourg limestone were used for this direct shear testing program. The cylindrical samples were drilled out of cubic limestone blocks using a 4’ Kitchen-Walker drill at Danton Machine & Welding Inc. in Kingston, Ontario, with diamond core bits from Bomba Diamond Tools in Scarborough, Ontario. Specifications of the Kitchen-Walker drill are listed in Table 8-1.
Table 8-1: Specifications of Kitchen-Walker drill used for drilling

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Year of manufacture</td>
<td>1971</td>
</tr>
<tr>
<td>Type of machine</td>
<td>E-3</td>
</tr>
<tr>
<td>Machine serial number</td>
<td>1827</td>
</tr>
<tr>
<td>Electrical supply 3 PH AC</td>
<td>550 Volts, 60 Hz</td>
</tr>
<tr>
<td>Full load current</td>
<td>6.8 Amps</td>
</tr>
<tr>
<td>Control circuit</td>
<td>550 Volts</td>
</tr>
</tbody>
</table>

The dominant orientation of intrablock structure is horizontal with some vertical connective segments around nodule boundaries, which is evident at the sample block scale (see Figure 8-7). When the blocks were wetted, preferential drying patterns emerged where the nodules dried first and the intrablock structure dried last. In other words, the preferential wetness of the intrablock structure shows the tendency of the intrablock structure to retain water (Figure 8-7 bottom). The top surface of the Cobourg limestone blocks were marked for drilling location and core bit diameters to be used (Figure 8-8). The core diameters selected for testing consist of 2 inch, 3 inch, and 4 inch diameters. The coring locations were first selected by diameter to meet the requirement for total length of samples. The placement of coring locations was selected to optimize the closest packing potential of used space (Figure 8-8c and d). In cases where a plane of weakness cut through the block, the coring locations were moved to accommodate the predicted zone of fracture (Figure 8-8c and d). Of the two blocks used, the first block did not have any visible planes of weakness and the geometrically optimal core locations were used. The second block had a semi-formed fracture through the middle (Figure 8-8c), so a modified drill pattern was used.
Figure 8-7: Examples of Cobourg limestone blocks from the Bowmanville quarry, showing the cross-section of intrablock structure on the vertical surfaces; the wet patterns on the bottom two blocks appeared during drying and highlight the uptake of water by the intrablock structure.
Figure 8-8: (a) Full scale drill pattern geometrically optimized to accommodate testing plan for 2”, 3” and 4” diameter cores; (b) drill pattern traced onto first Cobourg limestone block; (c) unique drill pattern for block developed with consideration of partially formed fracture plane to maximize intact drill cores; (d) drill pattern and fracture trace marked onto second Cobourg limestone block.
During drilling, the blocks were secured in place on the drill platform to minimize any disturbances caused by shaking, rotation, and oscillations (Figure 8-9a). City of Kingston tap water flowed through the drill bit during drilling to keep the contact area cool. The rotation and spindle speed of the drill was adjusted according to the rock type and core bit diameter to maximize the quality and smoothness of the circumferential surface of the rock cores. The most effective drilling speeds for this testing program are listed in Table 8-2.

### Table 8-2: Most effective rotational drilling speeds used for each diameter of core

<table>
<thead>
<tr>
<th>Core Diameter (inches)</th>
<th>Initial Drill Speed, (RPM) Used for first 1” of travel</th>
<th>Main Drill Speed (RPM) Used for remainder of travel (~15”)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>90</td>
<td>261</td>
</tr>
<tr>
<td>3</td>
<td>90</td>
<td>261</td>
</tr>
<tr>
<td>4</td>
<td>90</td>
<td>127</td>
</tr>
</tbody>
</table>

The accessible drilled rock cores were removed from the top of the block by hand and the remaining cores were removed after the fully drilled block was lifted with an overhead crane and friction fit strap (Figure 8-9b). For transport to the Queen’s University Geomechanics Laboratory, each piece of rock core was wrapped in several layers of plastic foam packing material (Figure 8-10). Separating each piece ensures the end fracture surfaces are preserved in their initial state for direct shear testing of existing fracture surfaces.
Figure 8-9: (a) Using Kitchen-Walker 4’ radial drill to extract a 3” diameter core out of a Cobourg limestone block; (b) Lifting remaining block, using an overhead crane and friction fit strap, off drill planform to collect exposed cylindrical cores

Figure 8-10: Packing process used for each individual piece of cylindrical core to preserve samples during transport from drilling site at Danton Drilling & Welding Inc. to Geomechanics testing laboratory at Queen’s University
8.4 Mineralogy of the Cobourg Limestone at the Bowmanville Quarry

The Cobourg limestone from St. Marys Cement Bowmanville quarry that was selected for this direct shear testing program is an analogue for the Cobourg limestone at the sited Bruce DGR repository depth of approximately 680 m below ground surface. In order to correlate the results of this testing program to investigations by NWMO on the direct shear mechanical properties of the Cobourg limestone at the Bruce DGR site at depth, it is important to define the mineralogical properties and differences of the Bruce DGR and the Bowmanville quarry limestones. The limestones were analyzed by X-ray diffraction (XRD) of powdered samples and Scanning Electron Microscopy (SEM) with associated Mineral Liberation Analysis (MLA) of polished thin sections. The rock samples used to make thin sections and powder were selected from the same offcut section of rock from the quarry block source of drill core for direct shear testing. The clay minerals are of particular interest to this investigation since they comprise a significant component of the dark grey layers (intragranular structure) between the calcite-rich nodules.

8.4.1 X-Ray Diffraction (XRD)

X-ray diffraction is a primary tool for identifying the mineralogy of powdered crystalline materials. The sample is powdered to homogenize the sample and to facilitate random orientations of particles on the measured surface. The X-ray diffraction properties of minerals are controlled by the crystal lattice structure, which will produce constructive interference from incident X-rays at specific angles that are controlled by the wavelength of the radiation, diffraction angle, and lattice spacing of the crystalline material (Bragg’s Law). Plotting the intensities of resultant diffracted X-rays with respect to a range of measurement orientations produces a characteristic pattern. The mineral constituents of the material can be identified by comparing the peaks to standard reference patterns (Parrish and Langford, 2006).

The powdered sample XRD tests were conducted using a Philips Panalytical X’pert Pro Multipurpose Diffractometer with an X’celerator detector. All of the samples were analyzed from 3-100 2θ using Co radiation (λ = 1.789 Å) for 90 s/step, and the results were analyzed using Panalytical Highscore Pro 4.0 software using the ICDD 2015 PDF 2 database. A baseline XRD analysis was conducted on two...
powdered Cobourg samples from the Bowmanville quarry. The results showed predominantly peaks representing quartz and calcite, with indistinguishable counts at low 2θ positions where clay mineral peaks typically occur (see Figure 8-11). The powder samples were treated with a hydrochloric acid solution until the calcite had fully reacted and reanalyzed with XRD. At this stage, the calcite peaks had significantly reduced, but the quartz signal dominated the results. To further refine the results, a clay separation procedure was conducted to isolate minerals by grain size. The four resulting fractions of the sample powder, with diameters of <2 μm, 2-5 μm, 5-10 μm, and >10 μm, were reanalyzed with XRD. The full XRD results after the clay separation procedure are shown in Figure 8-12 and a detail view of the primary clay peaks at less than 25° 2θ are shown in Figure 8-13. The XRD sample preparation procedure is described in more detail in Appendix C.

The peak identification of the clay separated sample indicates the probable major clay constituent to be illite. This mineral identification using the ICDD 2015 PDF 2 database is supported by the clay mineral identification flow chart by the United States Geological Survey (Poppe et al., 2001). To further refine this result, the sample was treated with ethylene glycol to test the expansion potential of the clay minerals, according to the procedure by Poppe et al. (2001). A detailed view of the <2 μm fraction of the calcite digested sample before and after treatment with ethylene glycol is shown in Figure 8-14. The peak at ~10° 2θ is unsymmetrical and there is a slight shift of the peak after the ethylene glycol treatment. According Poppe et al. (2001), these results suggest that the clay mineral constituent in the Cobourg limestone is interstratified illite-montmorillonite or illite-vermiculite.
Figure 8-11: X-ray diffraction results of the baseline test on the Cobourg limestone from the Bowmanville quarry
Figure 8.12: X-ray diffraction results of the sample fractions after digestion in hydrochloric acid and the clay separation procedure.
Figure 8-13: Detail view of the X-ray diffraction results of the sample fractions after digestion in hydrochloric acid and the clay separation procedure; the change in counts at the primary peak for illite and muscovite at ~10° 2θ are highlighted here.
Figure 8.14: Detail view of the X-ray diffraction results comparing the < 2 μm fraction of the calcite digested Cobourg limestone sample before and after treatment with ethylene glycol.
The XRD analyses of the Cobourg limestone selected for direct shear testing from the Bowmanville quarry identified the following constituent minerals: calcite, quartz, clinochlore, and interstratified illite-montmorillonite or illite-vermiculite. While the XRD method does not define the modal mineralogy of the samples, the baseline XRD results suggest that calcite is the major mineral constituent since its peaks have significantly higher counts. The subsequent digestion of calcite in hydrochloric acid and clay separation procedures revealed the clay minerals present in the sample. A significant amount of sample material was lost during the calcite digestion procedure, which is further evidence to support calcite as the major mineral constituent.

8.4.2 Scanning Electron Microscopy (SEM) and Mineral Liberation Analyzer (MLA)

Five polished thin sections of the Cobourg limestone (samples 0a to 0e) were cut from a variety of orientations to capture the heterogeneities and anisotropy of the rock. The dimensions of each thin section sample are approximately 20 mm wide and 30 mm long. The thin sections were coated with carbon for effective image capture with a back-scatter electron (BSE) detector and Energy Dispersive Spectrometry (EDS) analysis with a Bruker Xflash Si-drift detector on an FEI Quanta 650 FEG ESEM.

8.4.2.1 SEM and MLA Operating Conditions and Methodology

The images generated by the BSE detector are in greyscale, where the relative intensity (from 0-255) is controlled by the average atomic number of the phases in view. The SEM field emission electron gun generates an accelerated electron beam that travels through the sample. The interaction of the beam with the sample atoms produces elastic and inelastic collisions between electrons between electrons and atoms in the sample. Elastic collisions cause the beam electrons to alter their trajectory but experience no significant change in kinetic energy. Larger sample atoms are more likely to produce an elastic collision because of their larger cross-sectional area through which the beam electrons can interact. Consequently, sample phases that have a higher average atomic number record more elastic collisions, which results in more backscattered electrons. In terms of image intensity, more backscattered electrons correspond to a brighter intensity, while fewer backscattered electrons (caused by a travel through a sample phase with a
lower average atomic number) correspond to a darker intensity (Goedge, 2012). The SEM images in this study were calibrated to a copper standard to standardize the contrast and brightness between samples, and the beam settings were calibrated to achieve a beam current of 10 nA for optimal EDS analysis.

The EDS system on the SEM instrument was used for mineral identification. As the SEM electron beam interacts with the sample surface, energy is added to the atoms in the sample, which causes electrons in the inner shells to be ejected from the atom. Outer shell electrons move to the lower energy void in the inner shell, which then causes a characteristic X-ray photon to eject from the atom with an energy equivalent to the difference between the two shells. The EDS detectors count the amount of X-rays incoming at each energy, which results in an energy spectrum for each analysis location on the sample. Each element has characteristic energy peaks that can be used to identify minerals.

The MLA software controls the analysis locations on each sample and matches the detected energy spectra to a predetermined mineral reference library. With a grid of analysis locations, the MLA measurements across the grid provide a measurement of the modal mineralogy in each sample. The X-ray modal analysis (XMOD) classic point counting method is used in this study to generate a single X-ray analysis at each measurement point on the selected sample grid (Fandrich et al., 2007). The sample grid points for the XMOD analyses in this study are spaced every 50 pixels at a resolution of 700 × 700 px. The mineral reference library is composed of mineral names with the associated EDS energy spectra that occur in the sample. The mineral reference library is created using an XMOD_STD analysis of a subsection of the sample, where the unknown spectra from the grid measurement points are sorted into bins with common energy peaks. The sorted spectra bins were manually matched to known minerals using the procedure detailed by Severin (2004). For these tests, the mineral reference library was constructed using samples from the Bowmanville quarry as well as Cobourg samples from boreholes at the Bruce DGR site. The MLA mineral reference library created for this study is shown in Appendix D. The discussion of comparing the mineralogies between these source locations is discussed in the following section. The matching procedure relies on pre-existing knowledge of anticipated major mineral constituents in the sample for an accurate assessment. In some cases, an energy spectrum can include
multiple minerals, resulting in constructive or destructive interference on the spectrum, which can be problematic for mineral identification.

Once the mineral reference library is defined, XMOD analyses of the full samples are conducted, and each detected energy spectrum is compared to the reference library and assigned one of the mineral names. The energy spectrum must have a minimum best fit match of 70% to a spectrum in the mineral reference library to be assigned a mineral name. Measurement locations that do not match any documented spectra are labelled as unknown. A well-defined mineral reference library should have less than 2% of the measurements in a sample labelled as unknown (Buckwalter-Davis, 2013). Unknown spectra may be a result of several factors, including mineral intergrowth, an uneven mount surface, and small particle size. The results of these MLA analyses provide a measurement of the modal mineralogy in each thin section. This modal mineralogy is based on the area percentage of the identified grid points.

It is important to note that the EDS system and MLA results do not consider mineral crystal form nor differentiate mineral polymorphs. Minerals are identified during construction of the mineral reference library by their elemental composition. While MLA does provide a measurement of modal mineralogy, it is important to consider the results of the XRD analyses, which do consider crystal form, to improve the accuracy of the mineral constituent identification.

8.4.2.2 SEM and MLA Results

The modal mineralogies from the MLA analyses, based on the area percentage, are listed in Table 8-3. The unknown constituents did not match any energy spectra in the mineral reference library. The No_Xray constituents are sub-micron dark intensity grains where no X-ray was measured. The Low_Counts constituents are those spectra that did not have enough total X-ray counts to facilitate a successful pattern match.

The MLA analysis correlates well with the XRD analysis, where calcite is a dominant constituent. The other major constituent identified by the MLA analysis is a group of minerals that are too fine-grained to be detected individually. The elemental energy spectrum for this group contains interference
from multiple elements, where many are common to clay minerals. Based on the fine grain size and the elemental energy profiles, this group (termed clay matrix) is therefore considered to be clay-rich with fragments of other mineral constituents. The illite-vermiculite / illite-montmorillonite minerals identified in the XRD analysis are likely part of the clay matrix phase in the MLA results.

Further analysis of the thin section SEM images reveals the mineral geometry, crack initiators, and crack arresters (see Figure 8-15). Fossil fragments composed of calcite, ankerite, and sometimes gypsum are prevalent. Some fossil fragments are composed of apatite. The remaining mineral constituents mostly occur in the fine-grained matrix (e.g. Figure 8-15 a). The fossil fragments have well-preserved geometries and are, internally, relatively homogeneous (e.g. Figure 8-15 b and e). They have a range of porosities, and the pore space is sometimes infilled with pyrite (e.g. Figure 8-15 d and e). The stiffness contrast between the fossil fragments and the surrounding fine-grained matrix frequently results in cracks that propagate along those boundaries (e.g. Figure 8-15 b, c, and e). In some cases, pyrite has infilled linear zones that were likely pre-existing cracks (e.g. Figure 8-15 b and e). For cracks that breach into the fossil fragments, the geometry of the fragment tends to control the direction of crack propagation, which can redirect the crack from its orientation outside of the fossil fragment (Figure 8-15 c and Figure 8-16). The influence of larger fossil fragments on crack propagation may affect crack formation on a larger scale. This could explain the redirection of failure surfaces in the direct shear tests that formed above or below the identified zone of shear and intersected with the grout in the sample ring. Indeed, fossil fragments on pre-existing fracture surfaces in the core samples were observed to control and redirect the fracture surface into a more tortuous surface (Figure 8-17).
Table 8.3: Modal mineralogy of Cobourg limestone samples from the Bowmanville quarry determined by SEM and MLA analyses

<table>
<thead>
<tr>
<th></th>
<th>0a Area %</th>
<th></th>
<th>0b Area %</th>
<th></th>
<th>0c Area %</th>
<th></th>
<th>0d Area %</th>
<th></th>
<th>0e Area %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay matrix</td>
<td>51.85</td>
<td>Clay matrix</td>
<td>51.63</td>
<td>Clay matrix</td>
<td>52.53</td>
<td>Clay matrix</td>
<td>46.51</td>
<td>Clay matrix</td>
<td>50.02</td>
</tr>
<tr>
<td>Calcite</td>
<td>31.44</td>
<td>Calcite</td>
<td>31.19</td>
<td>Calcite</td>
<td>31.32</td>
<td>Calcite</td>
<td>40.84</td>
<td>Calcite</td>
<td>35.71</td>
</tr>
<tr>
<td>Ankerite</td>
<td>5.48</td>
<td>Ankerite</td>
<td>6.05</td>
<td>Ankerite</td>
<td>5.66</td>
<td>Ankerite</td>
<td>4.59</td>
<td>Ankerite</td>
<td>4.89</td>
</tr>
<tr>
<td>Unknown</td>
<td>2.47</td>
<td>Glaucosite</td>
<td>2.95</td>
<td>Quartz</td>
<td>2.36</td>
<td>Glaucosite</td>
<td>1.75</td>
<td>Quartz</td>
<td>2.10</td>
</tr>
<tr>
<td>Quartz</td>
<td>2.39</td>
<td>Quartz</td>
<td>2.53</td>
<td>Glaucosite</td>
<td>2.18</td>
<td>Unknown</td>
<td>1.72</td>
<td>Unknown</td>
<td>1.99</td>
</tr>
<tr>
<td>Glaucosite</td>
<td>2.29</td>
<td>Biotite</td>
<td>2.04</td>
<td>Unknown</td>
<td>2.00</td>
<td>Quartz</td>
<td>2.02</td>
<td>Glaucosite</td>
<td>1.70</td>
</tr>
<tr>
<td>Biotite</td>
<td>1.65</td>
<td>Unknown</td>
<td>2.03</td>
<td>Biotite</td>
<td>1.58</td>
<td>Biotite</td>
<td>1.39</td>
<td>No_Xray</td>
<td>1.24</td>
</tr>
<tr>
<td>Albite</td>
<td>1.01</td>
<td>Albite</td>
<td>0.91</td>
<td>Low_Counts</td>
<td>0.90</td>
<td>Albite</td>
<td>0.64</td>
<td>Biotite</td>
<td>1.15</td>
</tr>
<tr>
<td>No_Xray</td>
<td>0.82</td>
<td>Orthoclase</td>
<td>0.30</td>
<td>Albite</td>
<td>0.76</td>
<td>Orthoclase</td>
<td>0.20</td>
<td>Albite</td>
<td>0.66</td>
</tr>
<tr>
<td>Orthoclase</td>
<td>0.23</td>
<td>Pyrite</td>
<td>0.13</td>
<td>Orthoclase</td>
<td>0.24</td>
<td>Apatite</td>
<td>0.12</td>
<td>Orthoclase</td>
<td>0.18</td>
</tr>
<tr>
<td>Apatite</td>
<td>0.16</td>
<td>Apatite</td>
<td>0.12</td>
<td>Pyrite</td>
<td>0.16</td>
<td>Pyrite</td>
<td>0.08</td>
<td>Apatite</td>
<td>0.12</td>
</tr>
<tr>
<td>Pyrite</td>
<td>0.10</td>
<td>Gypsum</td>
<td>0.07</td>
<td>Apatite</td>
<td>0.12</td>
<td>Gypsum</td>
<td>0.07</td>
<td>Pyrite</td>
<td>0.11</td>
</tr>
<tr>
<td>Gypsum</td>
<td>0.06</td>
<td>Low_Counts</td>
<td>0.04</td>
<td>No_Xray</td>
<td>0.10</td>
<td>Low_Counts</td>
<td>0.03</td>
<td>Gypsum</td>
<td>0.09</td>
</tr>
<tr>
<td>Low_Counts</td>
<td>0.06</td>
<td>No_Xray</td>
<td>0.01</td>
<td>Gypsum</td>
<td>0.07</td>
<td>Sphalerite</td>
<td>0.01</td>
<td>Low_Counts</td>
<td>0.04</td>
</tr>
<tr>
<td>Sphalerite</td>
<td>0.00</td>
<td>Sphalerite</td>
<td>0.00</td>
<td>Sphalerite</td>
<td>0.00</td>
<td>No_Xray</td>
<td>0.03</td>
<td>Sphalerite</td>
<td>0.00</td>
</tr>
</tbody>
</table>
Figure 8-15: SEM images of Cobourg limestone thin sections from the Bowmanville quarry (30 m b.g.s.)
Figure 8-16: SEM image compilation of fossil-controlled crack propagation in a thin section of the Cobourg limestone from the Bowmanville quarry (30 m b.g.s.)
Figure 8-17: Example of fossil defining part of pre-existing fracture surface in drill core sample of Cobourg limestone from the Bowmanville quarry
8.5 Comparison of Cobourg Limestone Mineralogy between Bowmanville and Bruce Sites

Previous direct shear test results and some samples from testing of Cobourg limestone from the Bruce DGR exploratory boreholes were made available by NWMO for further examination. These direct shear tests were conducted by Canmet Mining and Mineral Science Laboratories in 2011 and the results are published in an NWMO technical report (Gorski et al., 2011). Spare offcuts of drill core from the previous geomechanical testing were preserved with the shear tested samples and source depth information.

Eight samples from a range of depths in the Cobourg formation were selected for mineralogical analysis. Seven are sourced from borehole DGR-5 and the eighth sample is sourced from borehole DGR-6. Four of these samples are from offcuts of previously tested direct shear samples, while the other selected samples are offcuts from other geomechanical test samples such as UCS and triaxial tests. The samples are described in Table 8-4. A photograph comparing Cobourg limestone from the Bowmanville quarry and the Bruce DGR boreholes illustrates the similarities between the samples, where there are light grey nodules and dark grey intrablock structures in between; however, the character of these components appears different upon initial inspection (Figure 8-18).

Table 8-4: List of DGR-5 and DGR-6 borehole samples tested for mineralogy

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Sample Source / ID</th>
<th>Depth b.g.s. (m)</th>
<th>Core run #</th>
<th>Unit</th>
<th>Correlation to Geomechanics Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>DGR5-700.65</td>
<td>700.65</td>
<td>168</td>
<td>Cobourg Fmn, Collingwood Member</td>
<td>Canmet Triaxial</td>
</tr>
<tr>
<td>2</td>
<td>DGR5-705.90</td>
<td>705.90</td>
<td>169</td>
<td>Cobourg Fmn, Collingwood Member</td>
<td>Canmet Direct Shear</td>
</tr>
<tr>
<td>3</td>
<td>DGR5-719.38</td>
<td>719.38</td>
<td>174</td>
<td>Cobourg Fmn, Lower Member</td>
<td>Canmet UCS/AEM</td>
</tr>
<tr>
<td>4</td>
<td>DGR5-719.65</td>
<td>719.65</td>
<td>174</td>
<td>Cobourg Fmn, Lower Member</td>
<td>Canmet Direct Shear</td>
</tr>
<tr>
<td>5</td>
<td>DGR5-732.20</td>
<td>732.20</td>
<td>178</td>
<td>Cobourg Fmn, Lower Member</td>
<td>Canmet Direct Shear</td>
</tr>
<tr>
<td>6</td>
<td>DGR5-735.61</td>
<td>735.61</td>
<td>179</td>
<td>Cobourg Fmn, Lower Member</td>
<td>Canmet UCS</td>
</tr>
<tr>
<td>7</td>
<td>DGR5-741.90</td>
<td>741.9</td>
<td>181</td>
<td>Sherman Fall Fmn.</td>
<td>Canmet Direct Shear</td>
</tr>
<tr>
<td>8</td>
<td>DGR6-750.99</td>
<td>750.99</td>
<td>193</td>
<td>Cobourg Fmn, Lower Member</td>
<td>Canmet UCS/AEM</td>
</tr>
</tbody>
</table>
The Bruce DGR limestone samples were analyzed using Scanning Electron Microscopy (SEM) of polished thin sections with associated Mineral Liberation Analysis (MLA). The mineral reference library for the MLA was constructed using thin sections from both the Bowmanville quarry and Bruce DGR Cobourg limestones. The modal mineralogies determined by the MLA in terms of area percentage for each sample 1 to 8 are listed in Table 8-5. Samples 3a and 3b are from the same rock sample but are in different orientations to capture some of the heterogeneities in the rock structure. A graphical comparison of modal mineralogies is shown in Figure 8-19. Calcite is the primary mineral constituent in all samples, followed by the clay matrix mineral group and ankerite as the other major mineral constituents. The remaining minerals in the reference library comprise up to 20% of the samples.
Table 8-5: MLA results of thin section analyses for samples from Bruce DGR boreholes

<table>
<thead>
<tr>
<th>1</th>
<th>2</th>
<th>3a</th>
<th>3b</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mineral Name</td>
<td>Area %</td>
<td>Mineral Name</td>
<td>Area %</td>
<td>Mineral Name</td>
<td>Area %</td>
<td>Mineral Name</td>
<td>Area %</td>
</tr>
<tr>
<td>Calcite</td>
<td>51.44%</td>
<td>Calcite</td>
<td>83.73%</td>
<td>Calcite</td>
<td>72.93%</td>
<td>Calcite</td>
<td>75.14%</td>
</tr>
<tr>
<td>Clay matrix</td>
<td>38.06%</td>
<td>Clay matrix</td>
<td>12.56%</td>
<td>Clay matrix</td>
<td>16.40%</td>
<td>Clay matrix</td>
<td>15.79%</td>
</tr>
<tr>
<td>Ankerite</td>
<td>6.52%</td>
<td>Unknown</td>
<td>1.92%</td>
<td>Ankerite</td>
<td>5.14%</td>
<td>Ankerite</td>
<td>4.22%</td>
</tr>
<tr>
<td>Unknown</td>
<td>1.52%</td>
<td>Quartz</td>
<td>0.45%</td>
<td>Unknown</td>
<td>4.06%</td>
<td>Unknown</td>
<td>1.51%</td>
</tr>
<tr>
<td>Quartz</td>
<td>0.86%</td>
<td>Ankerite</td>
<td>0.38%</td>
<td>Quartz</td>
<td>0.47%</td>
<td>Albite</td>
<td>1.35%</td>
</tr>
<tr>
<td>Biotite</td>
<td>0.47%</td>
<td>Gypsum</td>
<td>0.24%</td>
<td>Biotite</td>
<td>0.25%</td>
<td>Quartz</td>
<td>0.48%</td>
</tr>
<tr>
<td>Orthoclase</td>
<td>0.34%</td>
<td>Albite</td>
<td>0.19%</td>
<td>Orthoclase</td>
<td>0.24%</td>
<td>Gypsum</td>
<td>0.38%</td>
</tr>
<tr>
<td>Gypsum</td>
<td>0.33%</td>
<td>Low_Counts</td>
<td>0.14%</td>
<td>Albite</td>
<td>0.21%</td>
<td>Low_Counts</td>
<td>0.34%</td>
</tr>
<tr>
<td>Glaucnite</td>
<td>0.19%</td>
<td>Orthoclase</td>
<td>0.13%</td>
<td>Glaucnite</td>
<td>0.11%</td>
<td>Biotite</td>
<td>0.31%</td>
</tr>
<tr>
<td>Pyrite</td>
<td>0.13%</td>
<td>Pyrite</td>
<td>0.09%</td>
<td>Gypsum</td>
<td>0.08%</td>
<td>Orthoclase</td>
<td>0.21%</td>
</tr>
<tr>
<td>Apatite</td>
<td>0.12%</td>
<td>Biotite</td>
<td>0.08%</td>
<td>Pyrite</td>
<td>0.05%</td>
<td>Glaucnite</td>
<td>0.13%</td>
</tr>
<tr>
<td>Low_Counts</td>
<td>0.02%</td>
<td>Glaucnite</td>
<td>0.05%</td>
<td>Apatite</td>
<td>0.04%</td>
<td>Apatite</td>
<td>0.06%</td>
</tr>
<tr>
<td>Albite</td>
<td>0.01%</td>
<td>Low_Counts</td>
<td>0.02%</td>
<td>Pyrite</td>
<td>0.05%</td>
<td>Albite</td>
<td>0.05%</td>
</tr>
<tr>
<td>No_XRay</td>
<td>0.00%</td>
<td>No_XRay</td>
<td>0.00%</td>
<td>No_XRay</td>
<td>0.00%</td>
<td>No_XRay</td>
<td>0.01%</td>
</tr>
<tr>
<td>Sphalerite</td>
<td>0.00%</td>
<td>Sphalerite</td>
<td>0.00%</td>
<td>Sphalerite</td>
<td>0.00%</td>
<td>Sphalerite</td>
<td>0.00%</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>4</th>
<th>5</th>
<th>6</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mineral Name</td>
<td>Area %</td>
<td>Mineral Name</td>
<td>Area %</td>
</tr>
<tr>
<td>Calcite</td>
<td>75.53%</td>
<td>Calcite</td>
<td>68.72%</td>
</tr>
<tr>
<td>Clay matrix</td>
<td>17.70%</td>
<td>Clay matrix</td>
<td>15.60%</td>
</tr>
<tr>
<td>Ankerite</td>
<td>5.14%</td>
<td>Ankerite</td>
<td>6.09%</td>
</tr>
<tr>
<td>Unknown</td>
<td>0.61%</td>
<td>Biotite</td>
<td>2.26%</td>
</tr>
<tr>
<td>Quartz</td>
<td>0.40%</td>
<td>Unknown</td>
<td>2.11%</td>
</tr>
<tr>
<td>Orthoclase</td>
<td>0.21%</td>
<td>Albite</td>
<td>2.02%</td>
</tr>
<tr>
<td>Biotite</td>
<td>0.18%</td>
<td>Orthoclase</td>
<td>0.88%</td>
</tr>
<tr>
<td>Gypsum</td>
<td>0.08%</td>
<td>Quartz</td>
<td>0.88%</td>
</tr>
<tr>
<td>Glaucnite</td>
<td>0.06%</td>
<td>Glaucnite</td>
<td>0.79%</td>
</tr>
<tr>
<td>Apatite</td>
<td>0.03%</td>
<td>Low_Counts</td>
<td>0.21%</td>
</tr>
<tr>
<td>Pyrite</td>
<td>0.03%</td>
<td>Gypsum</td>
<td>0.20%</td>
</tr>
<tr>
<td>Low_Counts</td>
<td>0.02%</td>
<td>Pyrite</td>
<td>0.19%</td>
</tr>
<tr>
<td>Albite</td>
<td>0.00%</td>
<td>Apatite</td>
<td>0.04%</td>
</tr>
<tr>
<td>No_XRay</td>
<td>0.00%</td>
<td>No_XRay</td>
<td>0.00%</td>
</tr>
<tr>
<td>Sphalerite</td>
<td>0.00%</td>
<td>Sphalerite</td>
<td>0.00%</td>
</tr>
</tbody>
</table>
The modal mineralogies of samples 1 to 8 from the Bruce DGR boreholes are essentially constant between 700 and 750 m depths. When compared to the mineralogy of the Bowmanville quarry Cobourg limestone, however, there are changes in the proportion of the major mineral constituents: calcite and the clay matrix group (see Figure 8-20). The calcite content in the Bowmanville quarry Cobourg comprises approximately 50% of the samples, and increases to approximately 70% in the Bruce DGR samples. Conversely, the clay matrix content reduces from approximately 35% in the Bowmanville quarry Cobourg to approximately 20% in the Bruce DGR samples. There is no significant change in ankerite content between these sample locations, and the ankerite comprises approximately 5% of the samples.
The mineralogical variation between the Cobourg limestone from the Bowmanville quarry and Bruce DGR boreholes are primarily attributed to lateral variation in the depositional environment. There may be additional variation caused by the relative depth of formation. The two sites considered in this study are located on opposite sides of the basement topographic high Algonquin Arch, where the Bruce DGR site is on the flank of the Michigan Basin to the west and the Bowmanville quarry is located on the
flank of the Appalachian Basin to the east (see Figure 8-21). On a broad scale, the Michigan Basin is a relatively isolated intracratonic basin that experienced dominating carbonate deposition in the Ordovician, while the Appalachian foreland type Basin received more argillaceous sediment deposition in the Ordovician that originated from the activity of the Appalachian Orogen to the east (Gartner Lee Ltd., 2008b). This depositional history correlates well to the relatively higher clay matrix and lower calcite content in the Bowmanville quarry Cobourg limestone when compared to the Bruce DGR Cobourg.

The cracks observed in the SEM images of the samples from the Bruce DGR site exhibit similar behaviour to those observed in the surface samples. Fossil fragments are still present in the Bruce DGR samples, which sometimes control the direction of crack propagation to typically form along the contact
boundary between the fragment and surrounding matrix or through the homogeneous mineralogy of the fragment (Figure 8-22). In other areas of the sample, cracks preferentially travel through the fine grained clay matrix, with undulation in their paths caused by intersections with larger mineral grains such as calcite, quartz, ankerite, and pyrite. Two examples of this phenomenon are shown in Figure 8-23.

Figure 8-22: SEM image compilation of fossil-controlled crack propagation in a thin section of the Cobourg limestone from borehole DGR-5 at 732.20 m b.g.s.
Figure 8-23: SEM images of the Cobourg limestone from borehole DGR-5 showing the general grain distributions and cracks preferentially travelling through the fine grained matrix, with undulation caused by intersections with larger mineral grains (most commonly calcite)
8.6 Direct Shear Test Laboratory Procedure

The laboratory direct shear testing program of the Cobourg limestone included cylindrical core samples of 2 inch and 3 inch diameters, targeted testing zones of both pre-existing fracture surfaces and intact intrablock structure, and maximum constant normal stresses ranging from 0.2 to 8 MPa. While 4 inch diameter samples were part of the initial program design, there were not enough pre-fractured samples from the two source rock blocks, and upon further calculations, the predicted shear strength of intact intrablock samples could have exceeded the shear load capacity of the direct shear system.

The laboratory direct shear tests were performed with the GCTS RDS-200 Servo-Controlled Rock Direct Shear System (Figure 8-24), which features electro-hydraulic closed-loop digital servo control of the shear and normal loads for test automation. The system consists of:

- GCTS DSH-150 direct shear apparatus with a double acting +/- 100 kN capacity shear load actuator with 25 mm stroke and single acting 50 kN capacity normal load actuator with 25 mm stroke
- RDS-SERVOPAC hydraulic servo control package
- SCON-1500 microprocessor based digital servo controller and acquisition system
- DSH-330-150 sample specimen rings: each ring is 150 mm (6”) inside diameter, 178 mm (7”) outside diameter, and 76 mm (3”) high, and a pair of rings is required for a test
- RDS-GROUT quick set, non-shrinking grouting to affix rock sample inside shear box
- Computer with the GCTS controlling software
8.6.1 Sample Selection

The circumferential surface of cylindrical rock cores were photographically captured using a cylindrical scanner, which records an unrolled image of the surface. The scanner was specifically created for this study. The cylindrical scanner, designed and constructed in collaboration with Wesley Dossett (NSERC USRA), is shown in Figure 8-25. An example of an unrolled scanned image for a full 40 cm length of drill core is shown in Figure 8-26, and a detailed inset with labelled nodule, intrablock, and fracture features is shown in Figure 8-27. These images were used to measure the orientation of fracture and intrablock target shear surfaces with respect to the core axis for direct shear sample preparation.
Figure 8-25: Cylinder scanner used to capture the unrolled circumferential surface of a specimen (designed by Wesley Dossett, NSERC USRA)

The photographic levels of the scanned images were adjusted to exaggerate dark and light areas of the rock, which correspond to clay-rich intrablock and calcite-rich intact rock mineralogies, respectively (Figure 8-28b). The x-y coordinates of each target shear plane were graphed and a sinusoidal curve fit was generated and semi-automatically iterated until convergence of minimized residual values (Figure 8-28c). In cases where the automatic iteration produced results with more than one period, the initial fit parameters were adjusted to result in a best-fit to a single period around the circumference of the sample. When printed to scale and wrapped around the sample, the sinusoidal curves result in a best-fit plane through the sample. These sinusoidal curves and resulting planar fits were used to determine the bounding elliptical planes of the 1 cm-thick target shear zone and the total minimum height of the sample, approximately 6 cm.
Figure 8-26: Image from cylinder core scanner of the unrolled circumferential surface of 3 inch diameter core B1-3-4; the fracture surface between segments A and B was measured to determine the orientation of the fracture surface as a best fit ellipse with respect to the core axis for direct shear sample preparation; inset detail shown in Figure 8-27
Figure 8-27: Inset detail of unrolled core surface from cylindrical core scan shown in Figure 8-26.

Figure 8-28: (a) Original photographic scan of circumferential core surface; (b) photographic levels adjusted to amplify darker and lighter regions of core surface with selections of fracture and intact target shear planes; (c) sinusoidal curve fit of target shear plane.
8.6.2 Digital Preservation of Samples

Digital preservation of the shear surfaces as three-dimensional models was a key element of the sample preparation and testing procedure. A desktop laser scanner was initially used for the first suite of tests (2 inch diameter fracture samples). The surfaces of existing fracture planes were recorded using a NextEngine 3-Dimensional desktop laser scanner before and after shearing. The scanner records a 3D point cloud of the surface and a photographic image that is draped on the point cloud (Figure 8-29).

During testing, photogrammetry was explored as another option to record 3-Dimensional data of the shear test samples, and replaced the NextEngine scanning as the preferred method for the remaining tests. The relatively poor quality of the image overlay made by the desktop laser scanner is a significant limitation for the purposes of using the results to study the character of the shear surfaces. Photogrammetry with a high quality DSLR camera was found to produce much better quality images and the differences in 3-D point clouds are insignificant. A more thorough comparison of these technologies for 3D image capture of direct shear samples is beyond the scope of this study and is a topic of future research. The camera and sample setup for photogrammetry models requires 16 photographs taken at equal angles around the sample, which was facilitated by a consistently located tripod camera mount and a turntable for the sample. Photos for photogrammetry data were taken for each sample before testing (outside the grout and set in grout) and after testing (inside the grout). Vertical photographs of each sample were also taken immediately after the test for a photographic record. The photography setups are shown in Figure 8-30 and an example photogrammetry model is shown in Figure 8-31. Sample photographs were taken with a Canon 7D DSLR camera fitted with a compact-macro EF 50 mm prime lens. Photographs of each direct shear test sample are shown in Appendix A.
Figure 8-29. (a) Cobourg direct shear sample being scanned by NextEngine desktop 3-Dimensional laser scanner on rotating platform; (b) Photograph draped over 3D scan of fracture surface of direct shear sample; (c) 3D point cloud of sample generated by scanner
Figure 8-30: Camera setup for digital photograph collection; photogrammetry turntable at left and top right; vertical photos taken with colour correction palette for post-processing
8.6.3 Direct Shear Sample Preparation

Diamond saws were used to cut the direct shear samples. Water was used during the saw operation to maintain a cool working temperature on the blade and minimize airborne dust particles from the rock. The sinusoidal traces of elliptical planes were marked around the core circumference as saw cutting patterns (Figure 8-32). The saw cuts were made between 2.5 and 3 cm away from the target shear surface (Figure 8-33a and b). An example of a cut fracture sample is shown in Figure 8-33c. A similar tracing procedure was used for intrablock samples; in this case, the 1 cm thick target shear zone was best fit to centre the specifically targeted trace line of intrablock structure (Figure 8-34). A prepared intrablock sample is shown in Figure 8-35.
Figure 8-32. (a) Diamond saw used for cutting rock cores; (b) detail view of diamond saw blade aligned with designated drill plane

Figure 8-33. (a) Elliptical shear plane determined by circumferential intersection of sinusoidal best fit curve on paper wrapped around core; (b) Saw patterns marked parallel to elliptical target shear plane at 2.5-3 cm on each side of fracture plane; (c) 1 cm wide grout boundary marked parallel to elliptical target shear zone with a pre-existing fracture surface
Figure 8-34: New procedure for intrablock sample preparation to draw target shear zone on sample; (a-b) sinusoidal best fit of target shear surface ellipse, printed to scale and cut into bottom and top halves; (c) wrapping scaled sinusoidal best fit graph around sample; (d-f) tracing sine pattern onto sample surface so it is evenly distributed around the specifically targeted trace line of intrablock structure, with a spacing of 1.1 mm for the shear zone; here, the permanent marker used for tracing was approximately 0.5 mm thick, reducing the final zone to 1 cm
Figure 8-35: Example of a targeted intrablock structure shear surface with elliptical traces from the sinusoidal best fit curve, the 1 cm thick target shear zone (grout boundary), and alignment targets for photogrammetry model processing

Non-shrinking cement grout is used to hold the direct shear samples in sample rings during the test. In the laboratory, the dry grout mixture is hand mixed into water (Figure 8-36) immediately before pouring into a waiting shear ring (Figure 8-37). Several steps of the sample preparation process to install the sample into the sample rings are illustrated in Figure 8-37. The stainless steel sample rings holding the grout and sample have an outer diameter of 7 inches, inner diameter of 6 inches, and a height of 3 inches (Figure 8-38a). The bottom and top halves of each sample were encased in a steel sample ring. The sample halves were separated by a 1 cm thick layer of plasticine that was manually molded into place around the rock sample (Figure 8-38c). The plasticine is used to separate the sample and grout halves because it has significantly lower stiffness and strength than that of the rock sample and is therefore considered to have a negligible influence on the test. It is worthwhile to mention the inherent limitation of this direct shear test setup, where only target shear places with less than 1 cm undulation were candidates for testing. A number of possible Cobourg samples did not meet this criterion, which may bias the results. A detailed analysis of this possible phenomenon was beyond the scope of this project; however, the
author considers this to be an interesting topic for future research. A fully prepared direct shear sample is shown in Figure 8-38d. Here, the black collar is used to maintain the alignment of the sample while the grout cures. The collar is removed during the test so the only points of contact between the sample halves are the shear surface of the sample (load-bearing) and the spacer layer of plasticine (not load-bearing).

The prepared sample is then placed in the shear apparatus, the black steel collar is removed, and a level layer of angular construction sand from the top of the grout to the top of the sample ring is used to create a full and smooth contact surface with the cap (Figure 8-38e). For the test, the sample is covered with the cap that couples with the normal and shear load actuators (Figure 8-38f).

Figure 8-36: Cement grout materials and mixing process
Figure 8-37: (a) alignment of intrablock sample in shear ring; (b) bottom boundary of target shear zone is levelled with the top surface of the sample ring with a first pass by eye and confirmation by measurement with a level; (c) pouring grout into bottom sample ring while being careful to not get grout on the sample; (d) grout is filled to be flush with sample ring without spilling over; (e) 1 cm thick layer of plasticine is formed onto grout surface; (f) top sample ring is installed and held in place with black collar ring while grout is poured and cures
Figure 8-38: Sequence of preparation of direct shear sample rings; (a) bottom sample encased in grout and stainless steel ring; (b) detail view of grout surface aligned 0.5 cm below target shear plane; (c) top of 1 cm thick blue plasticine layer and upper steel ring in place; (d) fully prepared direct shear sample with grout in upper half and shear plane encased with temporary black collar; (e) shear sample in direct shear machine with collar removed and sand level used to have smooth contact with cap; (f) sample covered by cap that couples with normal and shear loads during test

8.6.4 Test Parameters
In all direct shear tests on the Cobourg limestone in this testing program, the normal stress, normal displacement, shear stress, and shear displacement are measured by the shear apparatus. All data points were recorded every 1 second. During a test, the normal load is increased to the desired value first while the normal displacement is recorded. This data is used to calculate normal stiffness properties of the sample. This normal loading procedure was cycled three times for fracture samples to measure the change
in normal stiffness with repeated loading. At the end of the first load increase (for intrablock samples) or third cycle (for fracture samples) when the normal load is at its maximum desired value for the test, the shear stage begins where the normal stress is maintained at a constant value and the upper half of the sample is sheared over the fixed lower half of the sample at a rate of 0.2 mm/minute (as suggested by ISRM, 1974) up to a displacement of 10 mm from the starting position. The shear stress and shear displacement data recorded during the shear stage of testing are used to calculate shear stiffness, peak shear strength, residual shear strength, and dilation angle properties.

8.6.5 Test Program and Schedule

This direct shear testing program investigates the effects of changing confining stresses and sample scales for tests through shear surfaces of interblock and intrablock structures. The target shear sample sizes have diameters of 2 and 3 inches. Pre-existing fracture surfaces (fracture samples) and intrablock structure (intrablock samples) were both tested. The normal stresses include 0.2, 0.5, 1.2, 2, 3, 5, and 8 MPa. The legend for sample identification numbers is shown in Figure 8-39 and the samples are listed in Table 8-6.

![Figure 8-39: Legend for direct shear sample identification numbers; Fbc refers to the interblock fracture surface between intact core pieces b and c of the 40 cm total length of 2 inch diameter core # 4 from block B1, which was used in the second 2 inch diameter fracture shear test at 0.5 MPa normal stress; I1 refers to the first target intrablock structure shear zone in the 3 inch diameter core # 1 from block B2, which was used in the second 3 inch diameter intrablock shear test at 8 MPa](image)
Table 8-6: List of direct shear test sample identification numbers and parameters

<table>
<thead>
<tr>
<th>#</th>
<th>Sample Diameter</th>
<th>Shear Interface</th>
<th>Normal Stress (MPa)</th>
<th>Sample ID (legend in Figure 8-39)</th>
<th>Test Date (DD/MM/YYYY)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2 inches (50.8 mm)</td>
<td>Fracture</td>
<td>0.5</td>
<td>B1-2-1-Fab-0.5a</td>
<td>26/02/2016 12pm</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.2</td>
<td>B1-2-3-Fab-1.2a</td>
<td>14/03/2016 9am</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2.0</td>
<td>B1-2-2-Fab-2a</td>
<td>10/03/2016 8:30am</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3.0</td>
<td>B1-2-1-Fbc-3a</td>
<td>11/03/2016 4pm</td>
</tr>
<tr>
<td>11</td>
<td></td>
<td></td>
<td>8.0</td>
<td>B1-2-2-Fbc-8a</td>
<td>11/03/2016 1pm</td>
</tr>
<tr>
<td>12</td>
<td></td>
<td>Intrablock</td>
<td>0.2</td>
<td>B1-2-4-B-I1-0.2a</td>
<td>18/05/2016 2pm</td>
</tr>
<tr>
<td>13</td>
<td></td>
<td></td>
<td>0.5</td>
<td>B1-2-2-A-I1-0.5a</td>
<td>14/05/2016 11am</td>
</tr>
<tr>
<td>14</td>
<td></td>
<td></td>
<td>1.2</td>
<td>B1-2-4-A-I1-1.2a</td>
<td>14/05/2016 12:30pm</td>
</tr>
<tr>
<td>15</td>
<td></td>
<td></td>
<td>2.0</td>
<td>B1-2-1-A-I1-2a</td>
<td>14/05/2016 14pm</td>
</tr>
<tr>
<td>16</td>
<td></td>
<td></td>
<td>3.0</td>
<td>B1-2-3-B-I1-3a</td>
<td>17/05/2016 9am</td>
</tr>
<tr>
<td>17</td>
<td></td>
<td></td>
<td>5.0</td>
<td>B1-2-4-B-I1-5a</td>
<td>23/05/2016 9am</td>
</tr>
<tr>
<td>18</td>
<td></td>
<td></td>
<td>8.0</td>
<td>B1-2-3-A-I1-8a</td>
<td>18/05/2016 12pm</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Fracture</td>
<td>0.5</td>
<td>A2-3-2-0.5a</td>
<td>06/04/2016 9am</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.2</td>
<td>A1-3-13-Fab-1.2a</td>
<td>10/04/2016 9am</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2.0</td>
<td>A2-3-6-Fab-2a</td>
<td>06/04/2016 12pm</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3.0</td>
<td>C2-3-2-Fab-3a</td>
<td>13/04/2016 9am</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>8.0</td>
<td>C2-3-6-Fab-8a</td>
<td>13/04/2016 12pm</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Intrablock</td>
<td>0.5</td>
<td>B1-3-1-I1-0.5a</td>
<td>12/07/2016 11am</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.2</td>
<td>B1-3-1-I2-1.2a</td>
<td>12/07/2016 1:30pm</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2.0</td>
<td>C2-3-B-I1-2a</td>
<td>12/07/2016 3pm</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3.0</td>
<td>B1-3-1-I3-3a</td>
<td>13/07/2016 9am</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>8.0</td>
<td>B2-3-4-I1-8a</td>
<td>13/07/2016 2pm</td>
</tr>
</tbody>
</table>

286
The direct shear tests were conducted from February to July 2016 in four major phases, which were organized by sample diameter and discontinuity type (pre-existing fractures versus intrablock structures). The overall schedule at the time of shear testing is illustrated in Figure 8-40. The full test process for a single sample requires two days of work, from sample selection in a cylindrical drill core to the direct shear test. By utilizing three pairs of sample rings, this process was optimized to conduct testing in batches of up to four samples at a time. The lower layer of grout was allowed a minimum of 2 hours to set before applying the 1 cm thick plasticine layer around the target shear zone. The plasticine application requires approximately 45 minutes per sample. Lastly, the upper layer of grout was allowed a minimum set time of 8 hours; this was typically scheduled overnight and the shear test was conducted the following day. The increase in testing rate through the program is largely attributed to continuous improvements in testing optimization and training of laboratory assistants.

![Testing schedule of the direct shear test program](image)

Figure 8-40: Testing schedule of the direct shear test program
8.7 Laboratory Direct Shear Test Results

The high level goals of this direct shear testing program are to determine mechanical stiffness and strength properties of the Cobourg limestone, with comparisons between pre-existing fractures and intrablock structure, 2 inch and 3 inch cylindrical sample diameters, and at a variety of normal stresses. All of the direct shear tests are constant normal load tests. Measurements of normal stiffness (\(K_n\)), shear stiffness (\(K_s\)), shear strength (Mohr-Coulomb criterion), and dilation parameters are discussed in the following sections. These established parameters for interblock structures are applied to the tests on intrablock structures in this investigation to provide a consistent basis for comparison. All test results are shown in Appendix B.

8.7.1 Pre-Peak Deformation Behaviour: Stiffness

Deformation properties for discontinuities were first introduced by Goodman et al. (1968) to quantify their pre-peak elastic behaviour for applications to numerical models that were beginning to use explicit structural elements. These stiffness properties are separated into normal stiffness (\(K_n\)) and shear stiffness (\(K_s\)) components, which can both be determined by means of laboratory testing.

8.7.1.1 Normal Stiffness

The normal stiffness component is measured during the initial stage of normal load application to the sample in a direct shear test, where the load is increased to the desired state for a constant normal load test. Very few measurements of normal stiffness for rock fractures are reported in the literature. Several researchers have proposed closure laws to quantify normal closure with respect to increasing normal stress. Goodman (1974) proposed an empirical hyperbolic fracture closure law between normal stress (\(\sigma_n\)) and normal displacement (\(\delta_n\)), where the non-linear curve tends to an asymptotic relationship at the maximum joint closure (i.e. normal displacement):

\[
\sigma_n = \frac{\Delta \delta_n}{\delta_{n\text{ max}} - \Delta \delta_n} \xi + \xi
\]  

(8-1)
Where $\Delta \delta_n$ is the change in normal displacement corresponding to an increase in normal stress from an initial seating pressure, $\xi$, to the value $\sigma_n$, and $\delta_{n_{\text{max}}}$ is the maximum joint closure. The normal stiffness, $K_n$, is therefore a variable equal to the derivative of this function.

Hungr and Coates (1978) performed direct shear tests on Ottawa limestone and sandstone sample and found linear fracture closure. This linear fit, instead of a hyperbolic fit, is a consequence of pre-compression in joints by far greater stresses in situ than what was applied in the laboratory tests. The maximum normal stress applied in the tests by Hungr and Coates (1978) is 2.3 MPa across $15 \times 15$ cm samples. The curvature of the hyperbolic fit proposed by Goodman (1974) is explained by the progressive crushing of previously intact asperities and the increase of the true contact area. Bandis et al. (1983) supported the hyperbolic relationship for application to mated fractures, but proposed a semi-logarithmic closure law to improve the quantification of non-mated fractures.

Zangerl et al. (2008) compiled laboratory and in situ experimental data of granite to determine a normal stiffness relationship for numerical modelling with explicit rockmass structure of the Gotthard highway tunnel in Switzerland. The maximum applied normal stresses included in this data set range from 1.4 to 160 MPa, with 92% of the data between 1.4 and 30 MPa. This study implemented the semi-logarithmic closure law for normal stiffness proposed by Bandis et al. (1983) to describe the non-linear behaviour of the data. The semi-logarithmic function was preferred by Zangerl et al. (2008) over the hyperbolic closure law since the relationship is defined by a single free parameter (instead of two free parameters for the hyperbolic function). Thus, the need for only one closure measurement over a small normal stress range to fully define the non-linear stiffness behaviour over a larger stress range is practically appealing for engineering applications (Zangerl et al., 2008). Evans et al. (1992) defined the single free parameter of the semi-logarithmic closure law as the “stiffness characteristic”, $dK_n / d\sigma_n$, where the change in normal displacement, $\Delta \delta_n$, resulting from a change in effective normal stress from an initial reference value, $\sigma_n^{\text{ref}}$, is given by
\[
\Delta \delta_n = \frac{1}{dK_n / d\sigma_n} \ln\left(\sigma'_n / \sigma_{n,ref}'\right)
\]  
(8-2)

For this semi-logarithmic closure law, the curve of normal stiffness versus effective normal stress is linear and passes through the origin, for zero normal stiffness at zero normal stress:

\[
K_n = \left(\frac{dK_n}{d\sigma'_n}\right)\sigma'_n
\]  
(8-3)

Therefore, the normal stiffness \(K_n\) can be obtained by multiplying any effective normal stress by the stiffness characteristic (Zangerl et al., 2008).

The normal stiffness of pre-existing fractures and intrablock structure in the tested Cobourg limestone samples has been analyzed with both linear and semi-logarithmic relationships between normal displacement and normal stress. The linear analysis follows from the findings by Hungr and Coates (1978) on Ottawa limestone, which is a similar rock type to the Cobourg limestone. The semi-logarithmic analysis follows from the findings by Zangerl et al. (2008) that provides an engineering approach to a non-linear stiffness relationship. The stiffness characteristic of each Cobourg sample for use in the semi-logarithmic closure law was measured as the slope of the linear best fit of normal displacement data with respect to \(\ln(\sigma_n)\). The normal stiffness for the linear closure law was measured as the slope of the linear best fit of normal displacement with respect to normal stress, \(\sigma_n\). Examples of these best fits are shown in Figure 8-41 and the results of each sample are shown in Appendix B.

A compilation of stiffness characteristics for all of the Cobourg limestone direct shear tests is illustrated in Figure 8-42. With increasing maximum normal stress, the stiffness characteristic tends to decrease for both sample sizes (2” and 3” diameter) and target shear zone types (pre-existing fracture and intrablock structure). This is a non-linear trend that may stabilize with additional testing at higher maximum normal stresses. The intrablock samples typically have higher stiffness characteristics than the fracture samples. The target shear zone in the intrablock samples have essentially full contact so the normal stiffness is dependent on the mineralogy and thickness of the intrablock structure compared to the wall rock. Conversely, the target shear surface in the fracture samples have less contact due to asperities.
and roughness characteristics of the fracture surface. The 2” diameter intrablock samples have the highest stiffness characteristic values, on average, while the 3” diameter fracture samples have the lowest values. There is no significant change in the stiffness characteristic between normal loading cycles in the fracture samples.

Figure 8-41: Example linear fits to three cycles of a fracture direct shear test that has a maximum normal stress of 8 MPa, with the semi-logarithmic and linear closure laws shown in the left and right graphs, respectively.

The stiffness characteristics of fractures and intrablock structures in the Cobourg were compared to the data on fractures in granite compiled by Zangerl et al. (2008), and illustrated in Figure 8-43. The Cobourg limestone stiffness characteristics fall on the lower bound of the granite data. This correlates with differences in wall rock intact strength between rock types. For example, representative UCS values
of 116 and 235 MPa have been reported for the Cobourg limestone (Jaczkowski et al., 2016) and Lac du Bonnet granite (Diederichs, 2007), respectively. Overall, the granite stiffness characteristic remains constant with increasing maximum normal stress. Further direct shear tests on the Cobourg limestone at increasing maximum normal stresses may result in a similar trend of stiffness characteristic that has little dependence on the maximum normal stress.

Figure 8-42: Compilation of stiffness characteristics for all Cobourg limestone direct shear tests

A compilation of normal stiffness results for all of the Cobourg limestone samples indicates a range of approximately 4 to 50 MPa/mm (4,000 to 50,000 MPa/m), with no dependence on normal stress, as shown in Figure 8-44. Results from both the linear and semi-logarithmic closure laws are included here. The semi-logarithmic normal stiffness values are the product of the stiffness characteristic and the maximum normal stress for each sample (as per Equation (8-3)).
Figure 8-43: Stiffness characteristics of pre-existing fractures and intrablock structure in the Cobourg limestone compared to the granite fracture database from Zangerl et al. (2008)

The linear normal stiffness values were measured as the slope of the linear best fit of normal displacement data with respect to normal stress, $\sigma_n$. Overall, the 3” fracture samples show the lowest normal stiffness values when calculated with the linear closure law. The semi-logarithmic 2” and 3” intrablock samples show the highest normal stiffness values.

The normal stiffness values for fracture samples range from approximately 4 to 30 MPa/mm (4,000 to 30,000 MPa/m), as shown in Figure 8-45. When comparing fracture samples only, normal stiffness values that were calculated using the semi-logarithmic closure law show higher values (approximately 9,000 to 30,000 MPa/m) than the linear closure law (approximately 4,000 to 10,000 MPa/m). There is a slight dependence of normal stiffness on maximum normal stress up to $\sigma_n = 3$ MPa.
The 2 inch diameter samples have higher normal stiffness values than the 3 inch diameter samples for both closure laws. The normal loading cycles have the least influence on normal stiffness, but may still produce variability on the order of several thousand MPa/m (see Figure 8-45).

Figure 8-44: Compilation of normal stiffness for all Cobourg limestone samples, including both linear and semi-log closure law results. The semi-logarithmic normal stiffness is the product of the stiffness characteristic and maximum normal stress for each sample.
Figure 8-45: Normal stiffness results for Cobourg samples with a pre-existing fracture, including both linear and semi-logarithmic closure law results. The semi-log normal stiffness is the product of the stiffness characteristic and maximum normal stress for each sample.

The normal stiffness values for intrablock samples range from approximately 5 to 50 MPa/mm (5,000 to 50,000 MPa/m), as shown in Figure 8-46. When comparing intrablock samples only, normal stiffness values that were calculated using the semi-logarithmic closure law show higher values (approximately 10,000 to 50,000 MPa/m) than the linear closure law (approximately 5,000 to 11,000 MPa/m), which is consistent with the results of the fracture samples. Similar to the fracture sample
results, there is a slight dependency of normal stiffness on normal stress, up to \( \sigma_n = 2 \) MPa. The 2 inch diameter samples have higher normal stiffness values than the 3 inch diameter samples for both closure laws, which is also consistent with the results of the fracture samples.

Figure 8-46: Normal stiffness results for intrablock structure Cobourg samples, including both linear and semi-log closure law results. The semi-logarithmic normal stiffness is the product of the stiffness characteristic and maximum normal stress for each sample.

A statistical analysis of the linear and semi-logarithmic closure laws was conducted to compare their suitability for use with this Cobourg limestone data. A quantitative statistical comparison using the coefficient of determination, \( R^2 \), was used to measure the variance of the best-fit lines to the data points. The \( R^2 \) values are higher for the linear closure law compared to the semi-logarithmic closure law (as
shown in Figure 8-47), which suggests the linear closure law is a better measure of normal stiffness for these direct shear test results.

A qualitative statistical analysis comparing the regular residuals of the best-fit lines for each closure law supports the result of the $R^2$ comparison, as shown in Figure 8-48. In this example of a fracture sample that has a maximum normal stress of 8 MPa, the linear closure law residual plot is more randomly dispersed around the horizontal axis when compared to the semi-logarithmic law, which indicates a better fit to the data. Note the $R^2$ values for the sample data (Figure 8-41) agree with the residuals analysis.

**Figure 8-47:** Quantitative statistical comparison using the coefficient of determination ($R^2$) to measure the variance of the linear regressions between the linear and semi-logarithmic closure laws to the normal stiffness data. This analysis suggests that the linear closure law is a better measure of normal stiffness for these direct shear test results.
Figure 8-48: Sample qualitative statistical comparison of the linear fit regular residuals between the linear and semi-logarithmic closure laws of cycle 1 shown in Figure 8-41; the linear closure law residual plot is more randomly dispersed around the horizontal axis when compared to the semi-logarithmic law, which indicates a better fit to the data. Note the $R^2$ values for the sample data agree with the residuals analysis.

The statistical analysis indicates the linear closure law is a better representation of the normal closure data to calculate normal stiffness. Therefore, the suggested ranges of normal stiffness values for these direct shear results of the Cobourg limestone should be reported based on results of the linear closure law. The suggested ranges of normal stiffness that are independent of normal stress are listed in Figure 8-42. The author considers these to be preliminary results; further refinement with additional testing at higher maximum normal stresses is strongly recommended to refine these values and to test for dependence of normal stiffness with increasing maximum normal stress. The normal stiffness data reported by Zangerl et al. (2008) shows non-linear normal closure is not well defined until a normal stress of approximately 20 MPa is reached (see Figure 8-49). Therefore, the author recommends further normal stiffness testing up to normal stresses of 20 MPa to see if the linear closure behaviour evolves to non-linear behaviour that may be better represented by the semi-logarithmic closure law.
Table 8-7: Results of suggested normal stiffness value ranges based on this direct shear test program of Cobourg limestone from the Bowmanville quarry

<table>
<thead>
<tr>
<th>Sample diameter</th>
<th>Normal stiffness range for Fracture samples (MPa/m)</th>
<th>Normal stiffness range for Intrablock samples (MPa/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 inch</td>
<td>6,000 to 10,000</td>
<td>5,000 to 10,000</td>
</tr>
<tr>
<td>3 inch</td>
<td>4,000 to 6,000</td>
<td>8,000 to 11,000</td>
</tr>
</tbody>
</table>

Figure 8-49: Laboratory and in situ normal closure data of granite. The red dashed threshold indicates maximum normal stress of approximately 20 MPa where non-linear behaviour fully develops (modified after Zangerl et al., 2008)

8.7.1.2 Shear Stiffness

The shear stiffness component is measured during the second stage of a constant normal load direct shear test, when the normal load has reached the desired level, and application of the shear load begins. Shear stiffness is measured as the slope of shear stress with respect to shear displacement between the onset of shear loading and the failure of the sample at the peak shear stress. According to the literature, there are two common approaches to measure shear stiffness: peak and yield. The peak shear stiffness is the most
common method and is defined as the secant between zero and peak shear stress. There is no consideration of the influence of sample seating in the non-linear portion near the beginning of the test, or non-linear behaviour that may occur immediately prior to reaching the peak shear stress. It is a simple method that is easy calculable by any practitioner. The yield stiffness is typically measured as the tangent at 50% of the peak shear stress, or approximated by the chord of the curve between 40% and 60% of the peak shear stress. In some cases, however, there are deviations from the elastic trend at these measurement points so the resulting shear stiffness may be inaccurate. The yield stiffness measurement is also a simple method that is easily measured with consistency by any practitioner. To address the mechanistic limitations with the existing techniques for measuring shear stiffness, a best fit chord measurement is proposed that requires manual selection of the linear elastic portion of the shear stress – shear displacement curve between the onset of shearing and the peak shear stress. This technique minimizes the influence of non-linear behaviour that commonly occurs early in the test and immediately before peak shear stress is achieved. This technique is admittedly subjective, but the mechanistic reasoning has merit for the effort required by the practitioner. Consistent measurements can be achieved through experience and developing an understanding of the typical behaviour of a given suite of test data. The three types of shear stiffness measurements are illustrated on a Cobourg limestone fracture sample shear test result in Figure 8-50. All three types of shear stiffness calculations were completed for each Cobourg limestone sample in this study, and the graphs and values for each sample are shown in Appendix B.
Figure 8-50: Measurement types for shear stiffness used in this study. Peak and yield types are found in the literature and the best fit chord type is a new approach developed in this testing program.

A database of literature data was compiled for this research to analyze reported trends of peak and yield shear stiffness values. This compilation of shear stiffness results with respect to the cross-sectional shear area is shown in Figure 8-51. Bandis et al. (1983) presented experimental evidence for a scale effect of peak shear stiffness, where peak stiffness decreases with increasing sample area. The peak shear stiffness data presented in Figure 8-51 supports this finding. The yield shear stiffness data, however, is significantly less scale dependent. The scale dependency introduced to peak shear stiffness is controlled by the non-linear portions of the shear stress – shear displacement loading curve. At the onset of shear stress, the ubiquitous non-linear portion is attributed to seating of the sample and shear test system. The non-linear behaviour immediately before peak is the deviation from elastic behaviour of the sample, with the onset of aperture crushing/grinding and local crack initiation for fracture samples. The shear testing of intrablock samples in this investigation also exhibits this non-linearity, which is attributed to crack initiation and development in the target intrablock shear zone.
The yield shear stiffness values from the literature range from approximately 500 to 40,000 MPa/m. This wide range is attributed to differences in wall rock type, the roughness of the fracture surface as defined by the distribution, amplitude, and inclination of asperities, and weakening infill materials such as clay minerals or other mineral coatings (Goodman et al., 1968).

![Graph showing shear stiffness results](image)

**Figure 8-51. Literature data of shear stiffness results from direct shear tests that has been compiled for this investigation to analyze global trends**

The shear stiffness values for all of the Cobourg limestone samples are dependent on normal stress, where shear stiffness increases with increasing normal stress. Overall, shear stiffness of the 2 inch diameter samples is more dependent on normal stress than the 3 inch diameter samples (see Figure 8-52 and Figure 8-53). In both sample sizes, the chord stiffness of the intrablock samples is the stiffest, while
the peak secant stiffness values of the fracture and intrablock samples are the softest. The 2 inch diameter samples show greater shear stiffness values when compared to the 3 inch diameter samples. Approximate ranges of the chord shear stiffness measurements for the entire range of tested normal stresses are listed in Table 8-8. While these ranges provide an approximation of shear stiffness, it is important to keep in mind the dependence to normal stress. Additional testing at higher normal stresses is recommended to investigate the continuation of this trend.

**Table 8-8: Results of suggested shear stiffness value ranges based on the chord type measurements for this direct shear test program of Cobourg limestone from the Bowmanville quarry**

<table>
<thead>
<tr>
<th>Sample diameter</th>
<th>Chord shear stiffness range for Fracture samples (MPa/m)</th>
<th>Chord shear stiffness range for Intrablock samples (MPa/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 inch</td>
<td>4,000 to 8,000</td>
<td>5,000 to 9,000</td>
</tr>
<tr>
<td>3 inch</td>
<td>3,000 to 4,000</td>
<td>4,000 to 5,000</td>
</tr>
</tbody>
</table>

To investigate the apparent scale effect of shear stiffness, ratios of stiffness for 3 inch / 2 inch diameter are shown in Figure 8-54. The ratio of shear stress stiffness is constant with normal stress, at approximately 0.6. The ratio of shear load stiffness between 3 inch and 2 inch diameter, is also constant with normal stress, and is approximately 1.25. The outliers at 2 MPa normal stress are due to the influence of one sample that had an essentially cohesionless response to shear loading. As a result, it is very soft (e.g. approximately 1,100 MPa/m secant shear stiffness in Figure 8-52). The outlier sample result is the only one of its kind in this testing program. After the test, inspection of the shear surface revealed a good quality test (with no grout interference), but there was very little contact area involved during shearing. A comparison of the shear displacement results of the outlier test (B1-2-2-Fab-2a) to the other 2 inch diameter fracture test at 2 MPa normal stress that had a typical response (B2-2-4-Fab-2b) is shown in Figure 8-55.

The apparent scale effect of chord stiffness observed in these test results is likely a result of the transition between 1<sup>st</sup> order small-scale roughness and 2<sup>nd</sup> order larger-scale undulations in the shear surfaces with sample upscaling from 2 inch diameter (~20 cm<sup>2</sup>) to 3 inch diameter (~45 cm<sup>2</sup>). The
wavelength of 2nd order undulations in the Cobourg intrablock structure and fractures is observed to be approximately 3 inches at the core and sample block scales (examples shown in Figure 8-56). Components of the undulations occasionally occurred in the 2 inch diameter fracture samples, but in those cases the shear directions were selected to avoid the high angle surfaces (e.g. Figure 8-56c). However, it is important to note that the nominal sample surfaces areas (20 and 45 cm²) of fracture samples are not entirely in contact and involved in shearing, so stiffness will depend only on the asperities in contact. At larger scales, the 2nd order undulations will be the controlling structure in a fracture surface, which are consistent up to at least the block sample size (40 cm × 40 cm, or 1600 cm² area) (see Figure 8-57). The undulation wavelengths shown in Figure 8-57 are traces of intrablock structure. This scale effect idea is applicable to both fracture samples and intrablock samples since most of the fracture samples tested in drill core are thought to have fractured through intrablock structure during the sample drilling process. This reasoning is discussed in more detail in the section on shear strength. When comparing the tested Cobourg chord stiffness values for fracture and intrablock samples to the literature database, the data falls in the middle of the literature data cluster for these sample sizes (see Figure 8-58). The yield shear stiffness literature data have little scale effect at sample areas greater than ~100 cm². Chord shear stiffness of the Cobourg limestone at larger scales are expected to behave similarly to the 3 inch diameter core since the 2nd order undulations will be the controlling asperity features in the shear zone.
Figure 8-52: Composite graph of shear stiffness results of 2-inch diameter fracture and intrablock samples
Figure 8-53: Composite graph of shear stiffness results of 3-inch diameter fracture and intrablock samples
Figure 8-54: Ratios of shear stiffness between 3 inch and 2 inch sample diameters; the left graph is the ratio between shear stress stiffness while the right graph is the ratio between shear load stiffness, which removes the surface area component of stress.
Figure 8-55: Outlier shear stiffness test (left) compared to the other 2 inch fracture test which has a typical result (right)
Figure 8-56: Example images of fracture shear surfaces before testing showing representative examples of roughness and undulation character; approximate shear directions are indicated by white arrows; (a-c) 2 inch diameter samples as 3D point clouds from a LiDAR scan; (d-f) 3 inch diameter samples in oblique photographs
8.7.1.3 Implications of Stiffness Property Selection on Excavation Response

The ranges of normal and shear stiffness determined through this direct shear testing program of the Cobourg limestone have implications in numerical modelling at the excavation scale when the rockmass structure is modelled explicitly. The responses of an excavation to explicit rockmass structure with different normal and shear stiffness properties are illustrated here using a Finite Element model of a panel access tunnel in the Bruce DGR with the geometry defined by the Mark II placement room design layout for sedimentary rock. The panel access tunnels have 25 m horizontal spacing (Radakovic-Gunzina et al., 2015). The intact rock properties for the Cobourg limestone were calculated using the brittle DISL.
criterion (Diederichs, 2007) and are listed in Table 8-9. The explicit structure included in this model for illustration purposes includes only horizontal persistent bedding with approximately 1 m spacing and non-persistent intrablock nodular structure with approximately 10-20 cm spacing and oriented parallel to the bedding.

Figure 8-58: Comparison of Cobourg (chord) shear stiffness results to literature (peak and yield) shear stiffness data
Figure 8-59: Bruce DGR Mark II placement room design layout for sedimentary host rock (Radakovic-Gunzina et al., 2015); the panel access tunnel is modelled in this study

Table 8-9: FEM model material stiffness and strength input properties

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Peak value</th>
<th>Residual value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young's Modulus (MPa)</td>
<td>40,000</td>
<td>N/A</td>
</tr>
<tr>
<td>Poisson's Ratio</td>
<td>0.2</td>
<td>N/A</td>
</tr>
<tr>
<td>UCS (MPa)</td>
<td>115</td>
<td>N/A</td>
</tr>
<tr>
<td>Hoek-Brown m</td>
<td>22</td>
<td>N/A</td>
</tr>
<tr>
<td>Cl (MPa)</td>
<td>45</td>
<td>N/A</td>
</tr>
<tr>
<td>Hoek-Brown a</td>
<td>0.25</td>
<td>0.75</td>
</tr>
<tr>
<td>Hoek-Brown m</td>
<td>0.52</td>
<td>8.00</td>
</tr>
<tr>
<td>Hoek-Brown s</td>
<td>0.0234</td>
<td>0.0054</td>
</tr>
</tbody>
</table>

Four models are used to vary the rockmass structure stiffness properties between maximum and minimum values. The normal and shear stiffness values are coupled such that the maximum normal stiffness is paired with the maximum shear stiffness for a given structure type. The strength properties of the rockmass structure are the same in all models to enable a direct comparison between varying normal and shear stiffness values without interference. The strength properties of the rockmass structure are
expressed by the Mohr-Coulomb failure criterion and are listed in Table 8-10. The overall model geometry is shown in Figure 8-60 and a detailed view of the excavation showing explicit structure and material mesh elements is shown in Figure 8-61.

Table 8-10: Mohr-Coulomb properties of rockmass structure

<table>
<thead>
<tr>
<th>Structure</th>
<th>Peak</th>
<th></th>
<th></th>
<th>Residual</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\sigma_l$ (MPa)</td>
<td>$c$ (MPa)</td>
<td>$\phi$ (°)</td>
<td>$\sigma_l$ (MPa)</td>
<td>$c$ (MPa)</td>
<td>$\phi$ (°)</td>
</tr>
<tr>
<td>Bedding</td>
<td>0</td>
<td>1.6</td>
<td>42.8</td>
<td>0</td>
<td>0.9</td>
<td>31.1</td>
</tr>
<tr>
<td>Intrablock</td>
<td>3</td>
<td>2.8</td>
<td>50.5</td>
<td>0</td>
<td>1.8</td>
<td>43.9</td>
</tr>
</tbody>
</table>

Figure 8-60: Finite element model geometry of DGR panel access tunnel in RS2 (RocScience, 2015)
The ranges of modelled normal and shear stiffness properties are listed in Table 8-11. The values reflect the results of the Cobourg direct shear tests discussed above. The normal stiffness values capture both the linear and semi-logarithmic values to reflect the possibility of a change in closure law to better fit the semi-logarithmic trend with analysis of tests at normal stresses greater than the maximum 8 MPa that was tested.

**Table 8-11: Ranges of modelled normal and shear stiffness properties**

<table>
<thead>
<tr>
<th>Structure Stiffness (MPa/m)</th>
<th>Bedding</th>
<th>Intrablock</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Normal, $K_n$</td>
<td>Shear, $K_s$</td>
</tr>
<tr>
<td>Minimum</td>
<td>4,000</td>
<td>3,000</td>
</tr>
<tr>
<td>Maximum</td>
<td>10,000</td>
<td>6,000</td>
</tr>
</tbody>
</table>
The model results are illustrated in Figure 8-62 comparing total displacement contours and yielded rockmass structure elements between the various cases of minimum to maximum stiffness values assigned bedding and intrablock structure. The yielded bedding and intrablock structure elements are comparable between models; however, there is a significant difference in total displacements at the excavation roof boundary, as shown in Figure 8-63. The case where the minimum stiffness values were assigned to the bedding and intrablock structure resulted in the most displacement in the roof, while the case of maximum stiffness values for both structures resulted in the least displacement. Although the finite element method is inherently a continuum where displacement magnitudes are not expected to directly reflect a real excavation, the comparison between models with different properties is still significant.

Further study is needed to investigate the implications of stiffness properties using fundamentally discontinuous discrete element models that allow block separation. In the DEM software by Itasca Consulting Group, UDEC, there is additional flexibility in the program that allows the user to program stress-dependent relationships, such as those found in the Cobourg limestone direct shear tests for normal and shear stiffness.
Figure 8-62: Total displacement around the excavation for each model with yielded structural elements in red
Figure 8-63: Comparison of total displacements in the excavation roof, moving upward away from the excavation boundary

8.7.2 Peak and Post-Peak Behaviour: Shear Strength

The peak shear strength is determined at the maximum shear stress the sample can sustain, for a given normal stress. After continued shear displacement, the residual shear stress that is sustained by the sample is the residual strength of the failure surface. Failure envelopes in normal stress – shear stress space can be defined with a collection of tests conducted at a variety of applied normal stresses. The linear Mohr-Coulomb failure criterion was selected for this study because the components of frictional and cohesive strength make it suitable for both interblock and intrablock structures. The non-linear Barton-Bandis criterion (Barton and Choubey, 1977) is not considered in this investigation because the lack of a term for cohesive strength limits its effective application to intrablock structure.
The conventional peak and residual stages of the Mohr-Coulomb criterion have been analyzed for the fracture samples. The three-stage Mohr-Coulomb approach developed in Chapter 6 was used in this study to analyze the intrablock direct shear sample data. The three-stage Mohr-Coulomb approach, with primary (peak), secondary (residual), and tertiary (ultimate) stages, is designed to improve the properties of strength behaviour for intrablock structure in numerical models with explicit rockmass structure. The primary (peak) stage considers intact intrablock structure so the stiffness and strength properties are related to the healed infill material. The secondary (residual) state is reached immediately after brittle failure, so the stiffness and strength properties should resemble a rough and clean fracture surface. The tertiary (ultimate) state represents the ultimate fracture condition after shearing, with smooth surfaces and possibly a mineral coating. Here, the strength properties resemble those of conventional residual fracture shear strength. An example of representative measurements of the peak, residual, and ultimate shear strengths of an intrablock sample are shown in Figure 8-64d. Shear strength measurements of all samples are shown in Appendix B.

The shear strength analyses of the test results are sorted by sample size. The results of all of the 2 inch and 3 inch diameter samples are shown in Figure 8-65 and Figure 8-66, respectively, including best fit linear failure envelopes defined by Mohr-Coulomb parameters. The data points that are highlighted in red are from samples that experienced grout interference during the post-peak stage of the shear test. Therefore, these post-peak results have been excluded from further analysis, and the filtered results for 2 inch and 3 inch diameter samples are shown in Figure 8-67 and Figure 8-68, respectively, including best fit linear envelopes defined by Mohr-Coulomb parameters. Nearly 50% of the shear surfaces intersected the grout during the post-peak stage of testing. In some cases the shear surface was dipping enough to cause the sample to ‘dive’ into the grout (see examples of grout interference in Figure 8-69). The fracture samples fared better than the intrablock samples. The alignment of shear surfaces for the fracture samples was relatively straightforward and in most cases the target fracture surface is where shearing occurred. However, the alignments of target shear zones for the intrablock samples were estimated based on qualitative observations of the intrablock structure around the circumference of the sample. The tortuosity
of the intrablock structure observed on the drill core and sample block surfaces suggests that the target shear zones internal to the samples may not have been planar. With this complication, however, several tests through intrablock structure survived into post-peak displacement without grout interference, which have been included in the shear strength results.

In both sample sizes, the peak failure envelope of the intrablock structure is stronger in both cohesion and friction strength components than the peak failure envelope of fracture samples. This is evidence to support the notion that intrablock structure is typically stronger than interblock structure. The residual and ultimate strength envelopes of intrablock structure show different behaviour between the sample sizes. In the 2 inch diameter samples, these envelopes suggest a cohesion-weakening, friction-strengthening behaviour that is used to describe brittle rock behaviour (e.g. Martin, 1997; Diederichs, 2007), but this behaviour is not present in the 3 inch diameter samples. In the 3 inch diameter samples, the intrablock sample residual strength envelopes are strain-weakening and are a good match to the fracture sample peak and residual envelopes. This suggests that the fracture surfaces may not be natural fractures, but would have been induced during the drilling process to extract core samples from the blocks. Therefore, the behaviour would follow the logic of the three-stage Mohr-Coulomb envelope for intrablock structure, where the intrablock residual envelope is defined as the strength of the fracture surface created immediately after failure but before significant shearing occurs. This result suggests that the apparent abundance of fractures found in the 40 × 40 × 40 cm blocks upon extracting the cylindrical samples may not be the case in the block as a whole, or in situ. This is a relevant observation for the context of DGR design where the abundance and frequency of in situ fractures is important for both the geotechnical representation of the rockmass for excavation design, and the hydrogeological representation of permeability and fluid flow, which is a key element that is used to predict potential flow of radionuclides through the rockmass.
Figure 8-64: Example measurements of peak, residual, and ultimate shear strengths for a good quality 2 inch diameter intrablock sample that was tested at a normal stress of 2 MPa; (a) overhead photo of the bottom half of the sample after shearing; (b) overhead photo of the top half of the sample after shearing; (c) an oblique view of the bottom half of the sample; (d) shear stress–shear displacement results that are used to measure shear stiffness and shear strength properties; (e) normal displacement–shear displacement results that are used to measure the post-peak dilation angle (dilation angle is discussed in the next section)
Figure 8-65: All shear strength results and best fit Mohr-Coulomb envelopes of the 2 inch diameter samples prior to discarding samples that experienced grout interference; the samples to be removed during data filtering are highlighted in red.
Figure 8-66: All shear strength results and best fit Mohr-Coulomb envelopes of the 3 inch diameter samples prior to discarding samples that experienced grout interference; the samples to be removed during data filtering are highlighted in red.
Figure 8-67: Shear strength results of the 2 inch diameter samples that have been filtered to exclude post-peak results that experienced grout interference.
Figure 8-68: Shear strength results of the 3 inch diameter samples that have been filtered to exclude post-peak results that experienced grout interference.
Figure 8-69: Examples of grout interference with post-peak shear behaviour: (a-b) 3 inch diameter intrablock sample B1-3-1-I3-3a tested at 3 MPa normal stress, where (a) is the top half and (b) is the bottom half; (c-d) 2 inch diameter fracture sample B1-2-4-Fab-3b tested at 3 MPa normal stress, where (c) is the top half and (d) is the bottom half. Zones of grout disturbance are indicated by the dotted lines in (b) and (c).

A further analysis to compare the Mohr-Coulomb friction angle and cohesion parameters for all strength envelopes is shown in Figure 8-70. Clusters of intrablock peak, fracture peak, and fracture residual strength envelopes are highlighted. Following the previous discussion, the error bars indicate the intrablock residual and ultimate envelopes for 2 inch diameter samples (that both have zero mean cohesion) have significant error, which suggests a poorer best fit line and therefore greater uncertainty.

The intrablock residual and ultimate envelopes for the 3 inch diameter samples fit within the clusters of fracture peak and residual strengths, respectively, and have significantly less error. This suggests that the three-stage Mohr-Coulomb may be a better model than a cohesion-weakening, friction-strengthening model. Since a significant number of tests results were filtered out of this analysis,
however, the author recommends that additional testing of intrablock structure, especially at higher normal stresses (greater than 3 MPa) is needed to refine these results.

![Figure 8-70: Summary of Mohr-Coulomb strength properties of all failure envelopes from the filtered data sets](image)

8.7.3 Post-Peak Behaviour: Dilation

Dilation is widely acknowledged as an important characteristic that is used to describe the post-peak behaviour of rock fractures (e.g. Barton, 1973; Hoek and Brown, 1997; Kaiser et al., 2000; Alejano and Alonso, 2005; Walton, 2014). The dilation process in brittle failing rock, also known as rockmass
bulking, is a result of new fracture growth, shear along existing discontinuities, and geometric incompatibilities when blocks of broken rock move into an excavation. These resultant large, permanent deformations in the fracture zone around an excavation boundary are detrimental to the continued effectiveness of ground support (Kaiser et al., 2000). Despite its influence on rockmass behaviour, dilation receives relatively little attention in routine engineering practice, which has been partially attributed to the general goal in rock engineering of avoiding rockmass failure (Alejano and Alonso, 2005). Constitutive dilatancy models for application in continuum numerical modelling comprise an active branch of geomechanics research (e.g. Alejano and Alonso, 2005; Zhao and Cai, 2010; Walton, 2014), which is beyond the scope of this research. However, the measurement of dilation angles of discontinuities in direct shear testing is an important part of characterizing the entire behaviour profile from pre-peak to post-peak for numerical models with explicit structure (Barton, 1973). In direct shear, dilation describes the progressive opening of the shear fracture caused by sliding over asperities during shear displacement. The strength of the rock material, angle of asperities, and applied normal stress contribute to the dilation angle (Goodman et al., 1968). The dilation angle represents the minimum energy path between shearing over asperities versus shearing through asperities, for a given normal stress. The maximum dilation angle for a fracture typically occurs at the instant after passing the peak shear strength (Barton, 1973). A greater level of normal stress and progressive shear displacement reduce the dilation angle as more asperities shear off the fracture wall during shear displacement (Barton, 1973; Alejano and Alonso, 2005). It is possible to have zero dilation with a perfectly smooth shear surface or with a normal stress high enough to crush the asperities.

The post-peak dilation angle in a direct shear test is determined during the final stage of the test, after shear failure, where the sample dilates (i.e. increases the normal displacement upward) during continued post-peak shear displacement. The measurement of dilation in this context is the rate of change of normal displacement with respect to shear displacement. The maximum dilation angle immediately after peak has been measured in this study. Corrections have been made for any dip of the failure surface from horizontal in the dilation angle measurement. An example of dilation angle measurement for a 3
inch diameter fracture sample under a normal stress of 1.2 MPa is shown in Figure 8-71. The dilation angle at peak shear strength is measured as the angle defined by the slope of normal vs. shear displacement initiating at the shear displacement corresponding to peak shear strength (38° in this example). The dilation results from each sample are shown in Appendix B. Photographs of this sample after testing are shown in Figure 8-72. This is a good quality failure surface because the fracture surface does not intersect the grout layers on either side of the 1 cm thick target shear zone. Therefore, post-peak behaviour was not influenced by any contact with the grout.

The failure surface was measured to have a 2° dip in the direction of shear after the test. The dilation angle was corrected based on the dip of the shear surface to represent the dilation angle on a horizontally oriented sample, giving a final corrected dilation angle of 36°. This correction procedure was done for all dilation angle results. In all cases, any dip of the shear surface was found to be dipping in the direction of shear. 3-dimensional photogrammetry models of the shear samples were used to measure the dips of the shear surfaces with respect to the sample ring as a horizontal reference plane. The plane orientation measurements were done using the IMInspect module of PolyWorks software by InnovMetric (2015) and a plane orientation macro script by R. Kromer (Ryan Kromer, personal communication, June 2016), as shown in Figure 8-73.
Figure 8-71: Example of the dilation angle measurement of a 3 inch fracture sample tested at 1.2 MPa normal stress (sample ID: A1-3-13-Fab-1.2a). The dilation angle is measured as the linear slope of normal displacement - shear displacement immediately after peak strength. The measured slope is indicated by green dashed line. The measured dilation angle was corrected based on the dip of the shear surface, which was measured on the sample after testing.
Figure 8-72: Photos of sample A1-3-13-Fab-1.2a after testing; (a) vertical view of bottom half of sample, which remained stationary during testing; (b) vertical view of top half of sample, which sheared to the right (toward the black arrow on the sample ring) with respect to the bottom half; (c) oblique view of bottom half of sample showing fracture surface does not intersect grout; (d) oblique view of top half of sample with plasticine showing fracture surface does not intersect grout
Figure 8-73: Three views of an example 3-Dimensional point cloud model of a direct shear sample in PolyWorks (InnovMetric, 2015) that was used to measure the orientation of the shear surface relative to sample ring (horizontal reference plane)

The compilation of dilation angle measurements for all shear tests is shown in Figure 8-74. The results of only 18 shear tests are included in this figure for two reasons. Firstly, the data collection for photogrammetry models began partway through the first phase of testing (2 inch diameter fracture samples); therefore the dips of shear surfaces could not be measured to appropriately correct the dilation angle data. Secondly, for the remaining samples, approximately 50% of the shear surfaces intersected the grout during the post-peak stage of testing. In accordance with the filtering conducted for the post-peak shear strength results, samples that experienced grout interference were discarded from the dilation angle analysis. The set of data suitable for dilation angle measurements includes 18 measurements, including multiple samples in each suite of tests.

Overall, the dilation angles decrease with increasing normal stress, which agrees with observations by Barton (1971) and others. The fracture samples have a lower dilation angle than the
intrablock samples, which agrees with the comparison between peak shear strength. The higher dilation angles of intrablock structure can be explained by the greater tortuosity observed in the intrablock structure at the core and sample block (40 × 40 × 40 cm) scales that would encourage the generation of an undulating fracture surface at peak shear stress. The abundant fossil content present in intrablock structure may contribute to a smaller order roughness of a fracture surface through intrablock structure. The dilation angles in both discontinuity types of 3 inch diameter samples are slightly greater than the 2 inch diameter samples. This is explained by the observed increase in undulation magnitude in the 3 inch diameter samples where more surface area captures the larger order wavelength of roughness. Quantification of the roughness of shear planes using the collected photogrammetry data is a topic of future research.

Figure 8-74: Dilation angle measurements of the Bowmanville quarry Cobourg limestone direct shear samples; Q2 (median) values are reported in the box plot
8.8 Comparison of Direct Shear Test Results between Bowmanville and Bruce Sites

The mineralogical results of the Cobourg limestone from the Bowmanville quarry and Bruce DGR boreholes DGR-5 and DGR-6 provide an explanation for differences between geomechanical laboratory direct shear test results of these rocks. The direct shear testing of the Cobourg limestone from the Bowmanville quarry conducted in this investigation is compared to results of testing the Cobourg limestone from the Bruce DGR boreholes in direct shear. The 3 inch diameter drill core from the Bruce DGR site was tested by Gorski et al. (2011) at Canmet Mining and Mineral Sciences Laboratories, Natural Resources Canada. These direct shear tests were conducted at constant maximum normal stresses between 1.4 and 3.0 MPa, which were converted from load using the nominal diameter of the samples. Linear normal stiffness, peak shear stiffness, and peak and residual shear strengths were reported. Four samples corresponding to the mineralogy samples discussed in the previous section were selected for analysis. The stiffness and strength properties of each sample are listed in Table 8-12 and photographs of the samples are shown in Figure 8-75.

Table 8-12: Direct shear results of borehole DGR-5 samples tested by Canmet

<table>
<thead>
<tr>
<th>Mineralogy sample #</th>
<th>Sample ID</th>
<th>Normal Stress (MPa)</th>
<th>Linear Normal Stiffness (MPa/m)</th>
<th>Peak Shear Stiffness (MPa/m)</th>
<th>Peak Shear Strength (MPa)</th>
<th>Residual Shear Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(2)</td>
<td>DGR5-705.90</td>
<td>1.4</td>
<td>8,510</td>
<td>4,528</td>
<td>4.02</td>
<td>1.56</td>
</tr>
<tr>
<td>(4)</td>
<td>DGR5-719.65</td>
<td>3.0</td>
<td>8,200</td>
<td>4,059</td>
<td>10.64</td>
<td>1.56</td>
</tr>
<tr>
<td>(5)</td>
<td>DGR5-732.20</td>
<td>1.4</td>
<td>8,750</td>
<td>3,563</td>
<td>4.87</td>
<td>1.49</td>
</tr>
<tr>
<td>(7)</td>
<td>DGR5-741.90</td>
<td>1.4</td>
<td>7,160</td>
<td>7,886</td>
<td>3.29</td>
<td>1.33</td>
</tr>
</tbody>
</table>
Figure 8-75: Photographs of core samples selected for direct shear testing at Canmet that are discussed in this investigation

These results were plotted with the test results of the Bowmanville quarry Cobourg limestone from this study on the normal stiffness, shear stiffness, and shear strength graphs (Figure 8-76, Figure 8-77, and Figure 8-78, respectively). The Canmet results for linear normal stiffness are stiffer than the results of the 3 inch diameter samples from this study, and nearly align with the semi-logarithmic results. Similarly for shear stiffness, the Canmet secant shear stiffness results are stiffer than the results from the Bowmanville quarry limestone by approximately a factor of two. In terms of strength, the peak shear strength results are stronger than the corresponding 3 inch fracture sample failure envelope from the Bowmanville quarry limestone. The residual shear strength results correlate well with the Bowmanville quarry fracture residual shear strength.
Overall, the Cobourg limestone samples from the Bruce DGR site are stiffer (in both normal and shear) and stronger than the Cobourg limestone samples from the Bowmanville quarry. This correlates to the mineralogical comparison, where the amount of coarse-grained calcite in the samples is greater at the Bruce DGR site and the weaker fine-grained clay matrix is correspondingly less abundant. This analysis presents a baseline correlation, supported by mineralogical reasoning, between mechanical properties of the Cobourg limestone in direct shear from the Bowmanville quarry to the Bruce DGR site.

Figure 8-76: Comparison of Canmet normal stiffness data to 3 inch fracture sample results from the Bowmanville quarry Cobourg limestone
Figure 8-77: Comparison of Canmet peak shear stiffness data to 3 inch sample results from the Bowmanville quarry Cobourg limestone
Figure 8-78: Comparison of Canmet peak and residual shear strength data to 3 inch sample results from the Bowmanville quarry Cobourg limestone
8.9 Discussion and Conclusions

This investigation of the Cobourg limestone characterized the mineralogical properties and mechanical behaviour in direct shear of the material. The Cobourg limestone has been sited to be the host rock for the Canadian nuclear waste deep geologic repository at the Bruce site near Kincardine, Ontario. The design life of the DGR is one million years in order to sufficiently isolate the waste radionuclide contaminants for their lifespan, which is the longest design life for any human engineered project. A prerequisite for the host rock where tunnels are constructed is stability in the geology, stress system, jointing, and faulting. It is therefore critical to determine geomechanical properties of the host rock for effective geotechnical design of the repository excavation. Direct shear tests were conducted to study the mechanical properties of both interblock and intrablock structure. The methodologies for strength and stiffness analyses for direct shear testing of joints was extended here to intrablock structure in order to develop streamlined direct shear test analyses across both interblock and intrablock types of rockmass structure. The conventional methods for measuring normal and shear joint stiffness were critically evaluated and multiple methods were applied to compare the results. A new method for measuring shear stiffness was proposed that uses mechanistic reasoning to target the elastic portion of the shear stress – shear strain curve. This approach is more subjective but with training it provides an improved representation of shear stiffness with relevance to numerical models with explicit rockmass structure.

Overall, the intrablock samples are stiffer and stronger than the fracture samples in direct shear, which supports the hypothesis of this thesis. Normal stiffness (linear) ranges from 4,000 to 10,000 MPa/m for fracture samples and 5,000 to 11,000 MPa/m for intrablock samples (including both 2 inch and 3 inch diameter sample sizes). Shear stiffness (chord) ranges from 3,000 to 8,000 MPa/m for fracture samples and 4,000 to 9,000 MPa/m for intrablock samples (including both sample sizes). A slight scale effect is observed in shear stiffness between 2 inch and 3 inch diameters, where the 3 inch diameter samples exhibit softer results. These scales represent a transition of behaviours caused by small-scale roughness of asperities and larger-scale undulations in the target shear surface. For larger diameter samples than what was tested, the larger-scale undulations are expected to become the dominant structure up to at least a
shear area of 40 cm × 40 cm, where the undulations were observed in the sample blocks. This consistency in the controlling surface features is expected to eliminate the apparent scale dependency found in these tests. This is supported by shear stiffness results found in the literature.

The shear strength results are expressed in terms of the Mohr-Coulomb failure criterion, since the cohesion term enables flexibility for use with intrablock structure. The shear strength of the fracture samples is comparable between sample sizes. The intrablock shear strength was expressed using the three-stage Mohr-Coulomb criterion discussed in Chapter 6, where the primary stage corresponds to peak strength of the intrablock structure intact material, the secondary stage is a residual strength immediately after brittle failure that resembles a clean, rough fracture surface, and the tertiary stage is an ultimate shear strength after shear displacement that corresponds to the residual strength of a fracture. The fracture sample peak and residual shear strength results are a good fit to the intrablock residual and ultimate strengths. This suggests that the fracture samples may be intrablock structure that fractured during the drill coring process but was intact in the sample block. The relative abundance of fracture samples recorded in the extracted drill core may therefore be an overestimate of the in situ fracture frequency parallel to bedding. This is a key observation in the context of DGR design, which relies on being situated in a low permeability rockmass that will act as a barrier for potential radionuclide transfer with fluid flow.

The measurements of dilation angle are greater for intrablock samples (~52° median) than fracture samples (~24° median). There is little difference between sample sizes. As expected, dilation angles tend to decrease with increasing stress, since the increased normal stress tends to prevent the shear surface from opening. Not all of the shear test results were included in the calculations for post-peak shear strength and dilation angle. Samples that were discarded from the analysis experienced interference with the grout during the test, where in most cases the fracture appeared to dive into the grout. The post-peak results for these samples would therefore be a measure of the rock-grout interaction.

The Cobourg limestone from the Bowmanville quarry that was selected for this direct shear testing program is an analogue for the Cobourg limestone at the Bruce DGR site and at a planned repository horizon depth of approximately 680 m below ground surface. In order to correlate the results of
this testing program to investigations by NWMO on the direct shear mechanical properties of the Cobourg limestone at the Bruce DGR site at depth, it is important to define the mineralogical properties and differences of the Bruce DGR and the Bowmanville quarry limestones. The limestones were analyzed by X-ray diffraction (XRD) of powdered samples and Scanning Electron Microscopy (SEM) of polished thin sections with associated Mineral Liberation Analysis (MLA). The clay minerals are of particular interest to this investigation, since they comprise a significant component of the intrablock structure in this rock that was targeted for direct shear testing of intrablock samples. The XRD analyses of the Cobourg limestone from the Bowmanville quarry identified the following constituent minerals: calcite, quartz, clinochlore, and interstratified illite-montmorillonite or illite-vermiculite. The clay minerals could only be identified after several sample treatment procedures to isolate the clay constituent: calcite digestion in hydrochloric acid, clay separation, and ethylene glycol treatment. The SEM and MLA analyses provided modal mineralogies of the Bowmanville quarry Cobourg limestone. The mineral constituents identified by MLA largely agree with the XRD results, but both tools were essential for the complete mineralogy characterization of the samples. The clay mineral constituents, for instance, were not specifically identified by the MLA.

Examination of the SEM images of the thin sections revealed a number of crack initiators and arresters in the material. The stiffness contrast between the fossil fragments and the surrounding fine-grained matrix frequently results in cracks that propagate along those boundaries. In some cases, pyrite has infilled linear zones that were likely pre-existing cracks. For cracks that breach into the fossil fragments, the geometry of the fragment tends to control the direction of crack propagation, which can redirect the crack from its orientation outside of the fossil fragment. The influence of larger fossil fragments on crack propagation may affect crack formation on a larger scale. This could explain the redirection of failure surfaces in the direct shear tests that formed above or below the identified zone of shear and intersected with the grout in the sample ring. Indeed, fossil fragments on pre-existing fracture surfaces in the core samples were observed to control and redirect the fracture surface into a more tortuous surface.
A comparison of mineralogies and direct shear tests between the Bowmanville quarry Cobourg limestone in this study and the Bruce DGR borehole samples of the Cobourg limestone tested in direct shear by Canmet revealed some differences between these source locations. From a mineralogy perspective of the two largest constituents, the amount of calcite in the samples increased and the amount of clay matrix material decreased toward the Bruce DGR site. This reflects the lateral variance in depositional environment between the intracratonic Michigan Basin to the west and the Appalachian Basin to the east that received more argillaceous material input from the Appalachian Orogen.

The cracks observed in the SEM images of the samples from the Bruce DGR site exhibit similar behaviour to those observed in the Bowmanville quarry samples. Fossil fragments are still present in the Bruce samples, which sometimes control the direction of crack propagation. A comparison of direct shear test results indicated an increase in normal stiffness, shear stiffness, and peak shear strength with depth. The residual shear strength from the Bruce DGR samples is a good match to the Bowmanville quarry results. This correlates with the observed variation in mineralogical comparison. This analysis presents a baseline correlation, supported by mineralogical reasoning, between mechanical properties of the Cobourg limestone in direct shear from the Bowmanville quarry to the Bruce DGR sample source locations.
8.10 References


Chapter 9

Discussion and Conclusions

9.1 Discussion

Geotechnical analysis for underground excavation design in complex geological rockmasses requires an increased understanding and more rigorous consideration of the impact of healed or “intrablock” structure, such as veins, on rockmass behaviour. Intrablock structure occurs between blocks of rock defined and bounded by “interblock structure”, the network of joints and other fractures conventionally considered in classic rockmass characterization, classification, and rockmass property estimation. When intersecting with modern excavations in deeper environments with higher and more complex stress paths, field observations have demonstrated that intrablock structure can have a significant influence on overall rockmass behaviour and should, therefore, be included in rockmass characterization and laboratory evaluation of mechanical properties.

This study and the resultant publications are dedicated to improving the understanding of the behaviour of intrablock structure at laboratory and excavation scales, and to develop new methodologies and tools to incorporate intrablock structure into rockmass characterization and geotechnical design practice. Within this context the outcomes of this thesis can be discussed within four main categories: (1) field characterization of complex rockmasses for numerical models with implicit structure; (2) brittle overbreak estimation for excavations in complex rockmasses; (3) characterization of complex rockmasses for numerical models with explicit structure; and (4) laboratory evaluation of intrablock structure for numerical models with explicit structure.

9.1.1 Characterization of Complex Rockmasses for Numerical Models with Implicit Structure

Field characterization of rockmasses is a key component of geotechnical design. Conventional design is typically conducted through the lens of rockmass classification systems such as RMR (Bieniawski, 1989) and Q (Barton et al., 1974), which have associated empirical guidelines for the design of ground support.
These tools resulted in a significant improvement to reliable geotechnical design before numerical tools were practically available. However, the limited input parameters result in output data that does not adequately capture the full impact of rockmass behaviour; nor can they be easily translated to numerical modelling parameters. Furthermore, these systems include little to no consideration of intrablock structure, and the databases of case studies that were used to develop these classification systems are from mining and tunnel cases from the mid-20th century that were not as deep as many modern excavations. For these reasons, conventional classification systems are not adequate for modern, numerical geotechnical design of complex rockmasses in deep settings where intrablock structure has an influence on the rockmass behaviour.

The discussions in Chapter 2 described the Geological Strength Index (GSI), which is a more flexible rockmass characterization tool that is tied into the Hoek-Brown strength criterion. The Hoek-Brown strength criterion is available in many modern geotechnical software packages to control the continuum material behaviour, so GSI is able to be a direct input to numerical models. GSI evaluates rockmass structure in terms of its geometry (block size) and discontinuity surface quality. It has been modified and quantified for improved use, and can be applied to any rockmass. For these reasons and as discussed in Chapter 3, GSI has been modified in this research to evaluate complex rockmasses that contain multiple suites of interblock and/or intrablock structures. Application of conventional characterization rationale to complex rockmasses would over-penalize the rockmass by using the worst case strength value; for example, by using the worst case in GSI of the structure and surface condition present. While intrablock structure can dominate the behaviour in a rockmass, it may not weaken the rockmass to that extent. Therefore, a new GSI chart has been created in this research to add a category in the discontinuity surface quality component for strengthening intrablock structure (reproduced in this chapter in Figure 9-1).

A key philosophy of the Composite GSI (CGSI) method is that it first assesses individual suites of rockmass structure by their GSI components before combining these values using a weighted harmonic average into a CGSI value that represents the rockmass as a whole. This result can be used as a direct
input to numerical models where the rockmass structure is considered implicitly using the Generalized Hoek-Brown strength criterion. Continuum modelling where rockmass structure is considered implicitly through strength criteria such as this is a preliminary step of numerical geotechnical design that is not computationally demanding and requires less input parameters than models with explicit rockmass structure. The CGSI method was also validated in Chapter 3 using a numerical test and a case study of an adit in a rockmass with hydrothermal intrablock structure at depths of up to 2000 m.

An alternative approach to characterize intrablock structure is from the laboratory testing scale of intact rock. The Generalized Hoek-Brown strength criterion (Hoek et al., 2002) was selected because it directly incorporates rockmass observations through the lens of GSI and the parameters have direct relevance to rock behaviour and input parameters for numerical models. The intact rock scale approach modifies the intact strength parameters of the material based on UCS, triaxial, and tensile laboratory tests. This approach allows laboratory strength testing programs to account for intrablock structure by using results where strength properties are influenced by the presence of veins instead of discarding them as erroneous. The case study of laboratory test results sorted by failure mode through the intact matrix versus intrablock structure was applied to an excavation scale numerical simulation to illustrate the effects on depth of rockmass yield. At a shallow depth (100 m), the modelled rockmass behaviour was similar for both the matrix and intrablock failure modes. At greater depths (500 to 1000 m), however, there is a significant difference in the extent of yield in both the material and explicit joint structure. These findings support the observations that indicate intrablock structures in complex rockmasses have limited influence in shallow (low stress) conditions but dominate behaviour at depth (high stress).
Figure 9-1: New GSI chart for complex rockmasses that contain intrablock structure. The added column is used to describe the infill quality of strengthening intrablock structure and descriptions of other intrablock structure have been added to existing columns. A summary of equations to calculate the Composite GSI (CGSI) is also provided.
9.1.2 Brittle Overbreak Estimation for Excavations in Complex Rockmasses

In sparsely jointed to massive hard rockmasses under high in situ and deviatoric stresses, brittle spalling is the dominant damage process and failure mode and can result in significant depths of overbreak outside the excavation design dimensions. An empirical tool with a mechanistic rationale for overbreak prediction in brittle rockmasses has been developed for homogeneous rockmasses. The study presented in Chapter 4 demonstrated that the empirical linear predictive trend for homogeneous rocks is ineffective for heterogeneous complex rockmasses. The mechanistic-based nonlinear predictive tools for different zones of failure in homogeneous rockmasses by Perras and Diederichs (2015) were applied to explain the behaviour of heterogeneous rocks, which captured the mean overbreak behaviour of each of the investigated units but not the maximum overbreak cases. Therefore, new predictive functions were created to fully capture the mean and overbreak cases for each unit.

The heterogeneous complex rockmass of focus in this study is in the New Mine Level project at the El Teniente Cu-Mo porphyry mine in Chile. The rockmass has very sparse jointing but there is pervasive intrablock structure in many lithological units. In addition to networks of intrablock structure, brecciated units are discussed as another type of heterogeneity. Observations of overbreak in the undercut level show the presence of brittle failure, and detailed overbreak profile measurements are applied to predictive depth of failure tools.

The investigation by Perras and Diederichs (2015) was driven by research for nuclear waste disposal, which is interested in individual excavation damage zones (EDZs). The highly damaged zone (HDZ) forms closest to the boundary and is characterized by instantaneous crack propagation and notch formation after initiation. The anhydrite breccia unit contains large clasts of contrasting mineralogy in the intact anhydrite and quartz matrix that act as crack arresters. The contact zone between the stockwork mafic complex and dacite porphyry shows similar overbreak patterns when compared to the anhydrite breccia since this contact zone in the investigated drift was likely mechanically brecciated by the intrusion of the dacite porphyry. These breccia units correlate well to the HDZ limit for homogenous rocks that only includes the immediate rupture after crack initiation.
The HDZ transitions to the inner EDZ (EDZ$_i$) which exhibits systematic and dilating damage, and will form a notch geometry with aggressive scaling. The EDZ$_i$ corresponds to the case histories based on maximum damage depth and empirical fit by Martin et al. (1999) and Diederichs (2007). The dacite porphyry unit has fewer stockwork veins than the stockwork mafic complex and the mean measured overbreak correlates well to the EDZ$_i$ limit for homogeneous rocks, which is the most likely limit of overbreak given time, no support, and aggressive scaling for average homogenous brittle rockmasses.

The EDZ$_i$ transitions to the outer EDZ (EDZ$_o$) which is characterized by partial connected to isolated damage and does not have significant dilation. The EDZ$_o$ is the limit of damage initiation which is normally minor in homogeneous rocks. However, this zone is relevant for heterogeneous rockmasses where pervasive veins, such as the quartz veins found in the complex mafic stockwork at El Teniente, act as crack attractors and allow for maximum crack propagation after initiation and immediate strength loss. This behaviour is correlated well to the EDZ$_o$ limit for homogeneous rocks. The overbreak data from the cases examined at El Teniente compared to the mechanistic EDZ failure prediction trends by Perras and Diederichs (2015) and the empirical trend by Diederichs (2010) and after Martin (1999) are reproduced here in Figure 9-2.

Although the means and single standard deviations are in reasonable agreement with the EDZ limits by Perras and Diederichs (2015), some of the maximum overbreak cases fall above the limits defined for their respective lithologies.
Figure 9-2: Ranges of overbreak depths for case drifts A and B at the El Teniente New Mine Level project. Means and standard deviations are plotted along with the absolute maximum overbreak measurements. Data is compared to the mechanistic prediction intervals by Perras and Diederichs (2015) for different parts of the excavation damage zone (EDZ) and linear empirical fit by Diederichs (2010) and after Martin (1999). The overbreak measurements from El Teniente vary by lithology with respect to the existing prediction functions due to the complex intrablock structures and other heterogeneous rocks that are present.

New functions for improved predictive ranges of maximum to minimum measured cases of brittle overbreak in the heterogeneous and complex stockwork mafic complex, dacite porphyry, and brecciated units are provided in Table 9-1 and illustrated in Figure 9-3 to account for all observed cases. The mechanistic framework of the functions developed by Perras and Diederichs (2015) has been preserved here, where the minimum case (zero) depth of spalling occurs at or below maximum tunnel stresses equal to crack initiation (CI).
Table 9-1: Maximum and minimum predictive functions for heterogeneous complex rockmasses based on observed brittle overbreak at the El Teniente New Mine Level Project

<table>
<thead>
<tr>
<th>Lithology</th>
<th>Maximum Predicted Overbreak</th>
<th>Minimum Predicted Overbreak</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stockwork Mafic Complex</td>
<td>$r / a = 1 + 0.95(\sigma_{\text{max}} / CI - 1)^{0.55}$</td>
<td>$r / a = 1 + 0.65(\sigma_{\text{max}} / CI - 1)^{0.5}$</td>
</tr>
<tr>
<td>Dacite Porphyry</td>
<td>$r / a = 1 + 0.85(\sigma_{\text{max}} / CI - 1)^{0.2}$</td>
<td>$r / a = 1 + 0.5(\sigma_{\text{max}} / CI - 1)^{0.4}$</td>
</tr>
<tr>
<td>Brecciated Units</td>
<td>$r / a = 1 + 0.35(\sigma_{\text{max}} / CI - 1)^{0.35}$</td>
<td>$r / a = 1 + 0.17(\sigma_{\text{max}} / CI - 1)^{0.65}$</td>
</tr>
</tbody>
</table>

Figure 9-3: Ranges of overbreak depths for case drifts A and B at the El Teniente New Mine Level Project, including new failure prediction ranges for these heterogeneous rockmasses defined by functions listed in Table 4-7. The range of “Brecciated Units” includes the anhydrite breccia and stockwork mafic complex – dacite porphyry contact units. The empirical linear fit by Diederichs (2010) and after Martin et al. (1999) is included for reference.
9.1.3 Characterization of Complex Rockmasses for Numerical Models with Explicit Structure

Numerical modelling for modern geotechnical design where rockmass structure is explicitly represented requires additional input parameters to describe their mechanical and geometrical properties, which have a critical influence on the modelled rockmass behaviour. Mechanical properties include deformation normal and shear stiffness as well as tensile and compressive strength, while geometric properties include orientation, spacing, and persistence of the structure.

Mechanical properties of intrablock structure including normal stiffness, shear stiffness, and strength, are the input requirements for rockmass structure in explicit numerical models. The fundamental approach in this thesis to address intrablock structure properties has been to extend the concepts from intrablock structure to maintain compatibility with numerical tools and to facilitate direct comparisons to interblock structures. Evaluations of intrablock mechanical properties have been conducted using numerical tools at the intact laboratory sample scale and at the excavation scale.

The study discussed in Chapter 5 focuses on extending joint stiffness and strength concepts to veins by calibrating numerical finite element simulations of Unconfined Compressive Strength (UCS) tests with explicit vein geometries that were determined by petrographic analysis of veins in thin section (Figure 9-4). Tracing the vein geometry with the minerals as intact material into a vein in a UCS model, when combined with stiffness and strength values of individual minerals reported in the literature, produced an axial stress-strain response. Next, the same UCS model but with the vein represented by a numerical structural element (the Goodman joint element) was used to calibrate stiffness and strength properties of the structural element to the stress-strain response of the first model. The resulting calibrated stiffness and strengths for structural elements was then applied to an excavation scale model to investigate the rockmass response of different vein properties that were based on a range of mineralogies present in different veins, including gypsum, quartz, pyrite, calcite, and muscovite. This methodology is a very useful approach for when physical laboratory testing may not be available, or at least as a preliminary assessment that can be subsequently validated by or adjusted using physical laboratory test results.
Figure 9-4: Intrablock structure property calibration at the laboratory test sample scale using thin section analysis; (top left) hand sample source of thin section of pyrite and muscovite vein; (bottom left) Thin section of vein with geometry of mineral grain boundaries constructed in FEM numerical model; (right) schematic of UCS simulation used to calibrate joint element stiffness and strength properties to axial stress-strain response of the discrete vein material model.

The study discussed in Chapter 6 addresses the selection and evolution of stiffness and strength values for the model discontinuity elements of intrablock structure. A new modification to the Mohr-Coulomb strength criterion is presented to better represent the behaviour of intrablock structure in explicit numerical models. This approach changes the stiffness and strength values of failed intrablock structural elements between pre-peak (“primary”), post-peak (“secondary”), and ultimate (“tertiary”) states (Figure 9-5). In the primary state when the intrablock structure is intact, the stiffness and strength properties are
related to the intact stiffness of the infilling material. In the secondary state just after brittle failure, the stiffness and strength values should be closer to those of interblock structure with rough and clean surfaces, as features of the new fracture surface will control the subsequent behaviour of the intrablock structure in the rockmass. The tertiary state represents a residual condition of the fracture surface after some shear displacement. This approach was tested using overbreak profiles of a deep drift in a hydrothermal setting with quartz vein intrablock structure and sparse jointing, and was found to provide a better match to the overbreak profiles when compared to the conventional two stage (peak and residual) Mohr-Coulomb criterion for fracture strength.

![Figure 9-5](image.png)

**Figure 9-5**: (a) Drill core section of wall rock with quartz vein; (b) primary (pre-peak), secondary (post-peak), and tertiary (ultimate) considerations of intrablock structure stiffness and strength

Geometric properties are most commonly, and in some cases exclusively, collected through drill core logging. The nature of data collected for conventional geotechnical characterization of drill core is often directed by inputs defined by empirical rockmass classification systems, such as Q (Barton et al., 1974) and RMR (Bieniawski, 1976, 1989). Since the time when these classification methods were developed, numerical modelling has become a very powerful and ubiquitous design tool. Core logging methods have not followed the same course of development for a variety of reasons, such as popularity, simplicity, and the desire in long term projects to correlate current data with older data. At this point,
most standard core logging practices do not capture the sophisticated data required for numerical input parameters. In the author’s opinion, this lack of sophisticated data is the primary impediment to more widespread use of explicit or discrete numerical modelling in geotechnical engineering practice. In comparison, computational hardware limitations to structural detail are a secondary challenge.

To this end, four core logging procedures with increasing levels of data capture were developed to compare the effectiveness of the interblock and intrablock structural data collected in each for geotechnical design decisions, as discussed in Chapter 7. The detail of these logging methods, from traditional rockmass classification parameters to current state of practice conventional logging and detailed intrablock data in oriented core, increases significantly, but the time required for logging also increases. While the models discussed in this study show that greater detail in logging results in more geologically accurate numerical models, there are consequences for the cost and schedule of a project. For more routine projects like mining where the excavation design life is relatively short term, method 3 (2nd most detail, 2nd longest time and cost) may be effective enough for numerical design inputs. For specialized geotechnical projects like DGRs for nuclear waste storage, method 4 (the most detailed, longest time and cost) may be necessary for the long term excavation life and publicly reviewed design. The parameters included in each core logging procedure are reproduced here in Table 9-2.
Table 9-2: Detailed parameters of four tested core logging methods including new methods 3 and 4

<table>
<thead>
<tr>
<th>Data Collected</th>
<th>Method 1</th>
<th>Method 2</th>
<th>Method 3</th>
<th>Method 4</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Drill Run Data</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>From (m), To (m)</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Core recovery</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td><strong>Rockmass Data</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strength estimate (R0-R6)</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Subordinate strength estimate</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>RQD</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Fracture spacing (m)</td>
<td></td>
<td></td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>Number of fractures</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Length of pieces</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Number of pieces</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total length</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td><strong>Structural Data</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Number of joint sets</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Joint roughness (Jr)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Joint alteration (Ja)</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Joint condition (Jcond)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Principal roughness</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Secondary roughness</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Joint Roughness Coeff. (JRC)</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Principal alteration</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Secondary alteration</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Principal infilling:</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>type, thickness (mm), quality</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Secondary infilling:</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>type, thickness (mm), quality</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Rock Data</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lithology</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Mineralization</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Alteration</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Phase</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Geotechnical unit</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td><strong>Intrablock Structure</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Block size mode (cm)</td>
<td>2b</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Type</td>
<td>2b</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Strength class (1, 2, 3)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Thickness (mm): max, min, mode</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Alteration halo (type)</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mohs hardness number</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Orientation Data</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Depth (m)</td>
<td>1b*</td>
<td></td>
<td></td>
<td>4b**</td>
</tr>
<tr>
<td>Alpha (degrees)</td>
<td>1b*</td>
<td></td>
<td></td>
<td>4b**</td>
</tr>
<tr>
<td>Beta (degrees)</td>
<td>1b*</td>
<td></td>
<td></td>
<td>4b**</td>
</tr>
</tbody>
</table>
| *For interblock structure, **For interblock & intrablock structure
Laboratory Evaluation of Intrablock Structure for Numerical Models with Explicit Structure

The concepts of normal stiffness, shear stiffness, and strength for discontinuities were developed before the inception of useful numerical tools and more so from scientific curiosity than practical design use. Early direct shear tests were conducted in the field for the direct assessment of specific, individual problematic discontinuities since they are expected to have the most significant impact on the rockmass strength (e.g. a clay filled gouge fracture underneath a dam foundation). These large scale tests are very expensive and it is therefore impossible to carry out a rigorous investigation of many discontinuities. Laboratory scale direct shear testing has become the preferred physical testing method to determine the mechanical behaviour of discontinuities. There is, however, little consistency between laboratory test reports of what parameters were measured and how they were calculated. This is particularly evident in the literature regarding normal and shear stiffness. Where reported values can be found with some effort, there are multiple measurement techniques reported. For normal stiffness, these include hyperbolic (Goodman et al., 1968), semi-logarithmic (Bandis et al., 1983; Zangerl et al., 2008), and linear (Hung and Coates, 1978) closure laws. For shear stiffness, these include peak secant and yield tangent measurements. Mohr-Coulomb shear strength values are more consistent in their measurements and more frequently reported. Dilation angle measurements that describe post-peak behaviour are not particularly common, but the measurement technique is fairly straightforward (e.g. Barton, 1973).

All of these mechanical parameters have been critically evaluated, from the perspective where the results will be used for explicit or discrete numerical modelling, for the direct shear testing program of the Cobourg limestone conducted for this thesis. This direct shear testing program includes pre-existing fractures (interblock) and intrablock structure. The Cobourg limestone is an argillaceous nodular limestone that has calcite-rich nodules and is surrounded by clay-rich layers that are considered to be intrablock structure (Figure 9-6). This formation has been sited to be the host rock for the Canadian nuclear waste deep geologic repository at the Bruce site in Kincardine, Ontario. The service design life for the DGR is one million years in order to sufficiently isolate the waste radionuclide contaminants for their lifespan, which is the longest design life for any human engineered project. A prerequisite for the
host rock where tunnels are constructed is stability in the geology, stress system, jointing, and faulting. It is therefore critical to determine geomechanical properties of the host rock for effective geotechnical design of the repository excavation. The methodologies for strength and stiffness analyses for direct shear testing of joints was extended here to intrablock structure in order to develop streamlined direct shear test analyses across both interblock and intrablock types of rockmass structure. A new method for measuring shear stiffness was proposed that uses mechanistic reasoning to target the elastic portion of the shear stress – shear strain curve. This approach is subjective but with training it provides an improved representation of shear stiffness with relevance to numerical models with explicit rockmass structure.

Figure 9-6: Unrolled image of drill core surface of the Cobourg limestone that was tested in direct shear, showing the heterogeneous nature of the calcite-rich nodules and clay-rich intrablock structure

The testing program included 2 inch and 3 inch diameter samples to investigate possible scale effects. A slight scale effect was observed in shear stiffness between 2 inch and 3 inch diameter, where the 3 inch diameter samples exhibit softer results. These sample scales represent a transition of behaviours caused by small-scale roughness of asperities and larger-scale undulations in the target shear surface. For
larger diameter samples than what was tested, the larger-scale undulations will become the dominant structure up to at least a shear area of 40 cm × 40 cm, where the undulations were observed in the sample blocks. This consistency in the controlling surface features is expected to eliminate the apparent scale dependency found in these tests. This is supported by shear stiffness results found in the literature.

The shear strength results are expressed in terms of the Mohr-Coulomb failure criterion, since the cohesion term enables flexibility for use with intrablock structure. The shear strength of the fracture samples is comparable between sample sizes. The intrablock shear strength was expressed using the three-stage Mohr-Coulomb criterion discussed in Chapter 6, where the primary stage corresponds to peak strength of the intrablock structure intact material, the secondary stage is a residual strength immediately after brittle failure that resembles a clean, rough fracture surface, and the tertiary stage is an ultimate shear strength after shear displacement that corresponds to the residual strength of a fracture. The fracture sample peak and residual shear strength results are a good fit to the intrablock residual and ultimate strengths. This suggests that the fracture samples may be intrablock structure that fractured during the drill coring process but was intact in the sample block. The relative abundance of fracture samples recorded in the extracted drill core may therefore be an overestimate of the in situ fracture frequency parallel to bedding. This is a key observation in the context of DGR design, which relies on being situated in a low permeability rockmass that will act as a barrier for potential radionuclide transfer with fluid flow.

The Cobourg limestone from the Bowmanville quarry that was selected for this direct shear testing program is an analogue for the Cobourg limestone at the sited Bruce DGR repository depth of approximately 680 m below ground surface. In order to correlate the results of this testing program to investigations by NWMO on the direct shear mechanical properties of the Cobourg limestone at the Bruce DGR site at depth, it is important to define the mineralogical properties and differences of the Bruce DGR and the Bowmanville quarry limestones. The limestones were analyzed by X-ray diffraction (XRD) of powdered samples and Scanning Electron Microscopy (SEM) of polished thin sections with associated Mineral Liberation Analysis (MLA). The clay minerals are of particular interest to this investigation, since they comprise a significant component of the intrablock structure in this rock that was targeted for
direct shear testing of intrablock samples. The clay mineral constituents identified in the Cobourg limestone using XRD are interstratified illite-montmorillonite or illite-vermiculite. A comparison of mineralogies and direct shear tests between the St. Mary Cobourg in this study and the Bruce DGR borehole samples of the Cobourg tested in direct shear by Canmet revealed some differences between the Cobourg near surface and at depth. From a mineralogy perspective of the two largest constituents, the amount of calcite in the samples increased and the amount of clay matrix material decreased with an increase in depth. A comparison of direct shear test results indicated an increase in normal stiffness, shear stiffness, and peak shear strength with depth. The residual shear strength at depth was a close match to the Bowmanville quarry Cobourg results. This correlates to the mineralogical comparison, where the amount of coarse-grained calcite in the samples increases with depth and the weaker fine-grained clay matrix decreases in abundance with depth. This analysis presents a baseline correlation, supported by mineralogical reasoning, between mechanical properties of the Cobourg limestone in direct shear from near surface to the repository depth.

9.1.5 Limitations of Current Research

The development of an understanding of intrablock structure in complex rockmasses presented in this thesis has included field observations and data collection, numerical analysis, and laboratory testing. The Finite Element numerical method used in this thesis can model rockmass structure as explicit elements effectively. However, the Finite Element Method is fundamentally continuum based, so fractures that yield cannot physically separate. While the Finite Element Method is regularly accepted in geotechnical modelling, comparisons to discontinuum methods such as the Discrete Element Method would enrich the findings of this research.
9.2 Summary of Conclusions

The research presented in this thesis covers a wide range of concepts that pertain to the behaviour of intrablock structure in complex rockmasses at depth. The primary conclusions of this research are summarized in the following sections.

9.2.1 Characterization of complex rockmasses for numerical models with implicit structure

- The new Composite GSI (CGSI) method was developed as a new method to assess the structural suites of rockmass structure individually before combining them into a weighted harmonic average value that describes the whole rockmass for input to equivalent continuum numerical models.

- The CGSI method was developed to account for both interblock and intrablock structure complex rockmasses in a way that does not underestimate rockmass strength. A numerical investigation and case study were used to compare the CGSI method to the conventional GSI approach and in both cases CGSI was found to be a more accurate representation of explicit rockmass behaviour.

- An optimization approach to assess whether to include rockmass structure in a numerical model implicitly or explicitly was presented for two case studies. The CGSI approach of considering suites of structure individually is advantageous for this optimization approach since individual suites of structure can be easily incorporated into either the implicit or explicit parts of the model.

- Intrablock structure can also be addressed at the intact laboratory sample scale, where failure through intrablock structure results in a weaker strength envelope when compared to failure through the intact rock matrix. Hoek-Brown material properties from the weaker strength envelope can be used as input to continuum rockmass material parameters.

9.2.2 Brittle overbreak estimation for excavations in complex rockmasses

- Brittle overbreak in hydrothermal complex rockmasses can be partially predicted using mechanistic curves developed for the different levels of excavation damage zones in homogeneous rockmasses by Perras and Diederichs (2015).
• The highly damaged zone (HDZ) forms closest to the boundary and is characterized by instantaneous crack propagation and notch formation after initiation. The observed breccia units contain large clasts of contrasting mineralogy from the matrix that act as crack arresters. These breccia units correlate well to the HDZ limit for homogenous rocks that only includes the immediate rupture after crack initiation.

• The HDZ transitions to the inner EDZ (EDZ_i) which exhibits systematic and dilating damage, and will form a notch geometry with aggressive scaling. The EDZ_i corresponds to the case histories based on maximum damage depth and empirical fit by Martin et al. (1999) and Diederichs (2007). The observed overbreak in the dacite porphyry unit that does not have abundant intrablock structure correlates well to the EDZ_i limit for homogeneous rocks.

• The EDZ_i transitions to the outer EDZ (EDZ_o) which is characterized by partial connected to isolated damage and does not have significant dilation. The EDZ_o is the limit of damage initiation which is normally minor in homogeneous rocks. However, this zone is relevant for heterogeneous rockmasses where pervasive veins act as crack attractors and allow for maximum crack propagation after initiation and immediate strength loss.

• A full capture of the mean and maximum brittle overbreak cases is accomplished using new functions that define maximum to minimum overbreak ranges for the hydrothermally altered stockwork mafic complex, dacite porphyry, and brecciated units from the New Mine Level Project at the El Teniente mine in Chile that were analyzed in this investigation.

9.2.3 Characterization of complex rockmasses for numerical models with explicit structure

• Numerical modelling for modern geotechnical design where rockmass structure is explicitly represented requires additional input parameters to describe their mechanical and geometrical properties, which have a critical influence on the modelled rockmass behaviour.
• Geometrical properties are most commonly evaluated by logging drill core. The standard practice of core logging uses inputs for empirical rockmass classification systems, which are not useful as inputs to numerical models with explicit structure.

• Four core logging methods were assessed (including two new ones that were created for this research) to compare the level of structural detail collected and its relevance to numerical modelling. A comparison of the amount of data collected to the quality of numerical model produced was weighed against the added time and cost of logging.

• A method to measure mechanical properties of intrablock structure was developed that extends joint stiffness and strength concepts to veins by calibrating numerical finite element simulations of Unconfined Compressive Strength (UCS) tests with explicit vein geometries that were determined by petrographic analysis of veins in thin section.

9.2.4 Laboratory evaluation of intrablock structure for numerical models with explicit structure

• Direct shear laboratory testing was conducted on fracture and intrablock samples in the Cobourg limestone.

• Laboratory testing procedures for fracture samples were successfully extrapolated and adapted to direct shear testing of intrablock samples.

• A linear closure law for normal stiffness of all tests was found to have the best fit for the data. Further testing at higher maximum normal stresses will have to continue analysis to determine if this trend becomes nonlinear.

• A new chord measurement technique for shear stiffness was created that requires manual selection of the linear elastic portion of the pre-peak shear stress – shear displacement curve that eliminates errors or bias from nonlinearities due to sample and equipment seating at the beginning of the test and immediately before peak. This is meant for input to numerical models where elastic curves are perfectly linear.
• The fracture sample peak and residual shear strength results are a good fit to the intrablock residual and ultimate strengths. This suggests that the fracture samples may be intrablock structure that fractured during the drill coring process but was intact in the sample block. The relative abundance of fracture samples recorded in the extracted drill core may therefore be an overestimate of the in situ fracture frequency parallel to bedding. This is a key observation in the context of DGR design, which relies on being situated in a low permeability rockmass that will act as a barrier for potential radionuclide transfer with fluid flow.

• Post-peak dilation results show that intrablock structure has a higher dilation angle than fracture samples.

• In order to correlate the results of this testing program to investigations by NWMO on the direct shear mechanical properties of the Cobourg limestone at the Bruce DGR site, it is important to define the mineralogical properties and differences of the Bruce DGR and the Bowmanville quarry limestones. The X-ray diffraction (XRD) testing of powdered Cobourg limestone from the Bowmanville quarry identified the following constituent minerals: calcite, quartz, clinochlore, and interstratified illite-montmorillonite or illite-vermiculite. Scanning Electron Microscopy (SEM) of polished thin sections with associated Mineral Liberation Analysis (MLA) provided modal mineralogies of the samples.

• Examination of the SEM images of the thin sections revealed a number of crack initiators and arresters in the material. The stiffness contrast between the fossil fragments and the surrounding fine-grained matrix frequently results in cracks that propagate along those boundaries. In some cases, pyrite has infilled linear zones that were likely pre-existing cracks. For cracks that breach into the fossil fragments, the geometry of the fragment tends to control the direction of crack propagation, which can redirect the crack from its orientation outside of the fossil fragment. The influence of larger fossil fragments on crack propagation may affect crack formation on a larger
scale. This could explain the redirection of failure surfaces in the direct shear tests that formed above or below the identified zone of shear and intersected with the grout in the sample ring.

- A comparison of mineralogies and direct shear tests between the Bowmanville quarry Cobourg limestone in this study and the Bruce DGR borehole samples of the Cobourg limestone tested in direct shear by Canmet revealed an increase in calcite content and a decrease in clay matrix content toward the Bruce DGR site. This correlates to an increase in stiffness and strength of the Cobourg in direct shear toward the Bruce DGR site. The difference in mineralogical composition can be attributed to the variation in depositional environments between the intracratonic Michigan Basin (Bruce DGR site) that is dominated by carbonates and the Appalachian Basin to the east (Bowmanville quarry) that accumulated more argillaceous material from the Appalachian Orogen source to the east.
9.3 Future Research

The research conducted for this thesis addresses many aspects of modern geotechnical design in complex rockmasses and improves the understanding of intrablock structure. This task involved extensive numerical simulations, field work, and laboratory testing. New research developments always come with new questions. There are several directions of research that can build on what has been investigated and accomplished in this thesis. Some of these directions for future research include the following:

- Application of the methods and results of intrablock characterization for explicit numerical modelling to discontinuous numerical codes such as the Discrete Element and Finite-Discrete Element Methods, as well as and three-dimensional numerical codes at both the laboratory sample and excavation scales.

- Investigation of the influence of small-scale roughness features in laboratory direct shear testing of fractures to assess what components of the shear surface within the nominal shear surface area are specifically involved in shearing and therefore the entire sample behaviour.

- The 3D digital photogrammetry data of direct shear sample surfaces before and after testing will be useful to create numerical simulations of the test and to calibrate the micromechanical parameters used in Discrete Element modelling to the physical laboratory test results.

- Expand the direct shear laboratory testing conducted on the sedimentary nodular intrablock structure to hydrothermal vein type intrablock structure and develop a database of stiffness and strength data for use in explicit or discrete modelling of intrablock structure.

- Investigation of the role of intrablock structures on post-peak rockmass behaviour with respect to geometric dilation.

- While success of the proposed characterization tools and numerical modelling techniques has been demonstrated using case studies, it is especially important in the field of geomechanics to apply these tools to cases in a variety of geological settings to realize their strengths and limitations. Therefore, further testing of the concepts and tools that were developed in this thesis is certainly encouraged.
9.4 Contributions

The scientific contributions developed through the research pertaining to this thesis are presented in Chapters 3 to 8. All contributions published or presented as part of this research are summarized below.

9.4.1 Articles Published in Refereed Journals


9.4.2 Articles Submitted for Review


9.4.3 Articles in Preparation


9.4.4 Fully Refereed Conference Papers (Day as first author only)


9.4.5 Refereed Extended Abstract and Presentation (no paper)


15. Day, J. J., Diederichs, M. S., and Hutchinson, D. J. 2013. The influence of intrablock structure on rockmass strength. 4th Canadian Young Geotechnical Engineers and Geoscientists Conference, cYGEKC, Mont-Tremblant, QC, October 3-6, 2 pages [oral presentation].
9.4.6 Reports Produced from Research Activities

9.4.7 Posters Produced from Research Activities

9.4.8 Invited Presentations
9.5 References


This page left blank
Appendix A

Direct Shear Test Sample Photographs
2 inch Diameter
Fracture
$B1-2-1-Fab-0.5a \quad \sigma_n = 0.5 \text{ MPa}$

Before Shearing

After Shearing

![Graph showing the model equation and data points](image-url)

<table>
<thead>
<tr>
<th>Model</th>
<th>Sine</th>
</tr>
</thead>
<tbody>
<tr>
<td>Equation</td>
<td>$y = y_0 + A \sin^2(\pi(x-x_c)/\lambda)$</td>
</tr>
<tr>
<td>Plot</td>
<td>B1-2-1-B-Fab</td>
</tr>
<tr>
<td>$y_0$</td>
<td>$-0.51965 \pm 0.95463$</td>
</tr>
<tr>
<td>$x_c$</td>
<td>$-3.44102 \pm 0.79948$</td>
</tr>
<tr>
<td>$\lambda$</td>
<td>$7.58604 \pm 0.36319$</td>
</tr>
<tr>
<td>$A$</td>
<td>$0.56264 \pm 0.00619$</td>
</tr>
<tr>
<td>Reduced Chi-Sqr</td>
<td>0.03079</td>
</tr>
<tr>
<td>R-Square(COD)</td>
<td>0.87007</td>
</tr>
<tr>
<td>Adj. R-Square</td>
<td>0.83463</td>
</tr>
</tbody>
</table>
$B1-2-4-Fbc-0.5b \quad \sigma_n = 0.5 \text{ MPa}$

**Before Shearing**

**After Shearing**

---

**Model**

**Equation**

$y = y_0 + A \sin(p(x-x_c))/w$

**Plot**

B1-2-4-C-Fbc

$y_0$  
$-0.17486 \pm 0.0199$

$xc$  
$-2.77786 \pm 0.00489$

$w$  
$7.71807 \pm 0.31466$

$A$  
$0.21292 \pm 0.02466$

**Reduced Chi-Sqr**  
$0.01242$

**R-Square(COD)**  
$0.71377$

**Adj. R-Square**  
$0.68515$
$B1-2-3-Fab-1.2a \quad \sigma_n = 1.2 \text{ MPa}$

Before Shearing

After Shearing

---

Model | $y = y_0 + A \sin \left( w (x - xc) / n \right)$
--- | ---
Plot | B1-2-3-A-Fab
$y_0$ | $0.33367 \pm 0.05195$
$xc$ | $6.14173 \pm 0.33304$
$w$ | $7.56515 \pm 0.27967$
$A$ | $0.05923 \pm 0.07339$
Reduced Chi-Sqr | 0.04493
R-Square (COD) | 0.65249
Adj. R-Square | 0.82088
Before Shearing

After Shearing

B2-2-5-Fab-1.2b \quad \sigma_n = 1.2 \text{ MPa}
$B1-2-2-Fab-2a \quad \sigma_n = 2.0 \text{ MPa}$

Before Shearing

After Shearing

---

<table>
<thead>
<tr>
<th>Model</th>
<th>Sine</th>
</tr>
</thead>
<tbody>
<tr>
<td>Equation</td>
<td>$y = y_0 + A \sin(p(x-x_c)) + e$</td>
</tr>
<tr>
<td>Plot</td>
<td>B1-2-2-A-Fab</td>
</tr>
<tr>
<td>$y_0$</td>
<td>-0.43742 ± 0.00396</td>
</tr>
<tr>
<td>$x_c$</td>
<td>101.41675 ± 30.2336</td>
</tr>
<tr>
<td>$p$</td>
<td>27.72214 ± 6.81793</td>
</tr>
<tr>
<td>$A$</td>
<td>0.25514 ± 0.10031</td>
</tr>
<tr>
<td>Reduced Chi-Sqr</td>
<td>0.13644</td>
</tr>
<tr>
<td>R-Square(COD)</td>
<td>0.23989</td>
</tr>
<tr>
<td>Adj. R-Square</td>
<td>0.14074</td>
</tr>
</tbody>
</table>
$B2-2-4-Fab-2b \quad \sigma_n = 2.0 \text{ MPa}$

**Before Shearing**

**After Shearing**

---

Model | Size
---|---
Equation | $y = A + B \cos(C(x - D))$
Plot | B2-2-4-A-Fab
$a$ | 0.23798 ± 0.01591
$b$ | 9.81844 ± 0.1305
$c$ | 7.50271 ± 0.1067
$d$ | 0.15097 ± 0.0202
Reduced Chi-Sqr | 0.00013
R-Square(COD) | 0.94827
Adj. R-Square | 0.94543

---

A-8
$B1-2-1-Fbc-3a \quad \sigma_n = 3.0 \text{ MPa}$

**Before Shearing**

**After Shearing**

---

**Model**

<table>
<thead>
<tr>
<th>Model</th>
<th>Sine Fit of Sheet1 AC1'B1-2-1-B-Fbc</th>
</tr>
</thead>
<tbody>
<tr>
<td>Equation</td>
<td>$y = y_0 + A \sin(\theta(x-x_c)/w)$</td>
</tr>
<tr>
<td>Plot</td>
<td>B1-2-1-B-Fbc</td>
</tr>
<tr>
<td>$y_0$</td>
<td>$-0.38037 \pm 0.02147$</td>
</tr>
<tr>
<td>$x_c$</td>
<td>$-4.08511 \pm 8.8576$</td>
</tr>
<tr>
<td>$w$</td>
<td>$7.5434 \pm 0.41622$</td>
</tr>
<tr>
<td>$A$</td>
<td>$0.21931 \pm 0.0282$</td>
</tr>
<tr>
<td>Reduced Chi-Sq$^2$</td>
<td>0.01163</td>
</tr>
<tr>
<td>R-Square(COD)</td>
<td>0.69027</td>
</tr>
<tr>
<td>Adj. R-Square</td>
<td>0.65709</td>
</tr>
</tbody>
</table>
$B1-2-4-Fab-3b$ \hspace{1cm} $\sigma_n = 3.0 \text{ MPa}$

**Before Shearing**

**After Shearing**

---

**Graph:**

- Model: Sine
- Equation: $y = y_0 + A \sin(\omega(x - x_c))/w$
- Plot: B1-2-4-B-Fab
- $y_0$: $-0.4763 \pm 0.04971$
- $x_0$: $-5.84165 \pm 1.24848$
- $w$: $7.03582 \pm 0.57181$
- $A$: $0.31965 \pm 0.06999$
- Reduced Chi-Sqr: 0.06814
- R-Square(COD): 0.43725
- Adj. R-Square: 0.37472
$B2-2-4-Fbc-3c$ \quad \sigma_n = 3.0 \text{ MPa}$

Before Shearing

After Shearing

\begin{align*}
\text{Model} & \quad \text{Sine} \\
\text{Equation} & \quad y = A \sin (B(x - C)) + D \\
A & \quad 0.3199 \pm 0.02497 \\
B & \quad 0.6294 \\
C & \quad 0.6294 \\
D & \quad 0.63308 \\
\end{align*}
$B1-2-2-Fbc-8a \quad \sigma_n = 8.0 \text{ MPa}$

Before Shearing

After Shearing

---

![Graph](image-url)

<table>
<thead>
<tr>
<th>Model</th>
<th>Sine Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>$y = y_0 + A \sin(\pi(x-x_c)/w)$</td>
<td></td>
</tr>
<tr>
<td>$P_{bt}$</td>
<td>$B1-2-2-B-Fbc$</td>
</tr>
<tr>
<td>$y_0$</td>
<td>$0.04808 \pm 0.00616$</td>
</tr>
<tr>
<td>$x_c$</td>
<td>$4.33344 \pm 0.25157$</td>
</tr>
<tr>
<td>$w$</td>
<td>$8.17427 \pm 0.29334$</td>
</tr>
<tr>
<td>$A$</td>
<td>$0.12317 \pm 0.0085$</td>
</tr>
<tr>
<td>Reduced Chi-Sqr</td>
<td>$0.0026$</td>
</tr>
<tr>
<td>R-Square (COD)</td>
<td>$0.70006$</td>
</tr>
<tr>
<td>Adj. R-Square</td>
<td>$0.69028$</td>
</tr>
</tbody>
</table>
$B1-2-3-Fbc-8b \quad \sigma_n = 8.0 \text{ MPa}$

Before Shearing

After Shearing

$\downarrow$

1.25x

1.25x

$\downarrow$

Model | Sine
--- | ---

| Equation | $y = y_0 + A \sin(\pi(x-x_c)/w)$ |
| Plot | $B1-2-3-C-Fbc$ |
| $y_0$ | $-0.29628 \pm 0.03183$ |
| $x_c$ | $-0.96609 \pm 0.37706$ |
| $w$ | $7.05383 \pm 0.18907$ |
| $A$ | $0.57511 \pm 0.04458$ |

Reduced Chi-Sqr 0.02681
R-Square(COD) 0.97935
Adj. R-Square 0.96395

A-13
2 inch Diameter
Intrablock
$B1-2-4-B-I1-0.2a \quad \sigma_n = 0.2 \, \text{MPa}$
$B2-2-5-B-I1-0.2b \quad \sigma_n = 0.2 \text{ MPa}$
B1-2-2-A-I1-0.5a \quad \sigma_n = 0.5 \text{ MPa}
$B1-2-4-C-I1-0.5b$ $\sigma_n = 0.5 \text{ MPa}$
$B1-2-4-A-I1-1.2a \quad \sigma_n = 1.2 \text{ MPa}$
\( B2-2-4-A-I1-1.2b \) \( \sigma_n = 1.2 \text{ MPa} \)
B1-2-1-A-I1-2a \quad \sigma_n = 2.0 \text{ MPa}
B2-2-5-A-I1-2b \quad \sigma_n = 2.0 \text{ MPa}
**B1-2-3-B-I1-3a**  \[ \sigma_n = 3.0 \text{ MPa} \]
\( B2-2-5-B-I3-3b \quad \sigma_n = 3.0 \, \text{MPa} \)
$B2-2-4-B-I1-5a \quad \sigma_n = 5.0 \text{ MPa}$
$\sigma_n = 8.0 \text{ MPa}$
3 inch Diameter
Fracture
\( \sigma_n = 0.5 \, \text{MPa} \)
\( C2-3-1-Fab-0.5b \quad \sigma_n = 0.5 \text{ MPa} \)
\[ A1-3-13-Fab-1.2a \quad \sigma_n = 1.2 \text{ MPa} \]
$\sigma_n = 1.2 \text{ MPa}$
A2-3-1-Fab-2b \quad \sigma_n = 2.0 \text{ MPa}
C2-3-2-Fab-3a  \[ \sigma_n = 3.0 \text{ MPa} \]
$C2-3-13-Fbc-3b \quad \sigma_n = 3.0 \text{ MPa}$
C2-3-6-Fab-8a \hspace{1cm} \sigma_n = 8.0 \text{ MPa}
$C2-3-9-Fbc-8b$  $\sigma_n = 8.0 \text{ MPa}$
3 inch Diameter
Intrablock
$B1-3-1-I1-0.5a$  \hspace{1cm} \sigma_n = 0.5 \text{ MPa}$
$B1-3-3-I3-0.5b \quad \sigma_n = 0.5 \text{ MPa}$
$B1-3-1-I2-1.2a \quad \sigma_n = 1.2 \text{ MPa}$
$B1-3-3-I2-1.2b \quad \sigma_n = 1.2 \text{ MPa}$
C2-3-B-I1-2a  \( \sigma_n = 2.0 \text{ MPa} \)
$B1-3-6-I1-2b$  \hspace{1cm}  \sigma_n = 2.0 \text{ MPa}$

![Image of test samples](image1.png)

![Image of test results](image2.png)

![Graph of sine fit](image3.png)

<table>
<thead>
<tr>
<th>Model</th>
<th>Sine</th>
</tr>
</thead>
<tbody>
<tr>
<td>Equation</td>
<td>$y = y_0 + A \sin(p(x-x_0) + w)$</td>
</tr>
<tr>
<td>Plot</td>
<td>B1-3-6-I1</td>
</tr>
<tr>
<td>$y_0$</td>
<td>0.36983 ± 0.04049</td>
</tr>
<tr>
<td>$x_0$</td>
<td>5.2261 ± 0.07921</td>
</tr>
<tr>
<td>$w$</td>
<td>11.65592 ± 0.21772</td>
</tr>
<tr>
<td>$A$</td>
<td>0.34523 ± 0.05189</td>
</tr>
<tr>
<td>Reduced Chi-Sqr</td>
<td>0.02362</td>
</tr>
<tr>
<td>R-Square(COD)</td>
<td>0.74445</td>
</tr>
<tr>
<td>Adj. R-Square</td>
<td>0.86853</td>
</tr>
</tbody>
</table>
B1-3-2-I1-3b  \quad \sigma_n = 3.0 \text{ MPa}
$B2-3-3-I1-5a \quad \sigma_n = 5.0 \text{ MPa}$
$\sigma_n = 5.0 \text{ MPa}$
B2-3-4-I1-8a \quad \sigma_n = 8.0 \text{ MPa}
$B2-3-1-I1-8b \quad \sigma_n = 8.0 \text{ MPa}$
Appendix B

Direct Shear Test Data
2 inch Diameter
Fracture
B1-2-3-Fab-1.2a  $\sigma_n = 1.2 \text{ MPa}$

**Shear Stiffness**
- Secant = 4157 MPa/m
- Tangent = 5762 MPa/m
- Pseudo-secant = 6154 MPa/m
- Peak = 3.90 MPa
- Residual = 1.76 MPa

**Dilation Angle ($\gamma'$)**
- At Peak Shear Strength = N/A
- Dip of Shear Surface = N/A
- Corrected Dilation Angle = N/A

**Normal Stress vs. Normal Displacement**
- Cycle 1: $y = 6.3729x + 0.085$, $R^2 = 0.9980$
- Cycle 2: $y = 6.6560x - 0.019$, $R^2 = 0.9978$
- Cycle 3: $y = 6.5983x - 0.047$, $R^2 = 0.9976$

**Shear Stress vs. Shear Displacement**
- Peak Shear Stress

**Bottom half post-shear**
- Top half post-shear
B1-2-2-Fab-2a \( \sigma_n = 2 \text{ MPa} \)

- **Shear Stiffness**
  - Secant = 1107 MPa/m
  - Tangent = 3123 MPa/m
  - Pseudo-secant = 3442 MPa/m

- **Shear Strength**
  - Peak = 2.58 MPa
  - Residual = 2.42 MPa

**Shear Displacement, \( \delta_s \) (mm)**

- **Dilation Angle (\( \psi \))**
  - At Peak Shear Strength = N/A
  - Dip of Shear Surface = N/A
  - Corrected Dilation Angle = N/A

**Normal Stress, \( \sigma_n \) (MPa)**

- \( y = 5.519x + 0.018 \)  \( R^2 = 0.9920 \)

- \( y = 6.928x - 1.445 \)  \( R^2 = 0.9331 \)

**Bottom half post-shear**

**Top half post-shear**
**B2-2-4-Fab-2b**  \( \sigma_n = 2 \) MPa

(a) Shear Stress, \( \tau \) (MPa)

Shear Stiffness
- Secant = 3189 MPa/m
- Tangent = 4484 MPa/m
- Pseudo-secant = 4704 MPa/m

Shear Strength
- Peak = 2.74 MPa
- Residual = 2.07 MPa

(b) Shear Stress, \( \tau \) (MPa) vs. Shear Displacement, \( \delta_s \) (mm)

(c) Normal Displacement, \( \delta_n \) (mm) vs. Shear Displacement, \( \delta_s \) (mm)

Dilation Angle (\( \Psi \))
- At Peak Shear Strength = 18°
- Dip of Shear Surface = 4°
- Corrected Dilation Angle = 14°

(d) Normal Stress, \( \sigma_n \) (MPa) vs. Normal Displacement, \( \delta_n \) (mm)

(e) \( \ln(\sigma_n) \) vs. Normal Displacement, \( \delta_n \) (mm)

- Cycle 1: \( y = 6.7502x + 0.046; R^2 = 0.9929 \)
- Cycle 2: \( y = 7.2794x - 0.205; R^2 = 0.9899 \)
- Cycle 3: \( y = 7.4324x - 0.318; R^2 = 0.9913 \)

Bottom half post-shear  Top half post-shear
Shear Stress, $\tau$ (MPa)

- Peak Shear Stress = 3.70 MPa
- Residual = 2.52 MPa

Shear Stiffness
- Secant = 3742 MPa/m
- Tangent = 3547 MPa/m
- Pseudo-secant = 4997 MPa/m

Dilation Angle ($\psi$)
- At Peak Shear Strength = N/A
- Dip of Shear Surface = N/A
- Corrected Dilation Angle = N/A
**B2-2-4-Fbc-3c**  \( \sigma_n = 3 \text{ MPa} \)

---

**Shear Stiffness**

- Secant: 5003 MPa/m
- Tangent: 6834 MPa/m
- Pseudo-secant: 6778 MPa/m
- Residual: 2.86 MPa

**Shear Strength**

- Peak: 5.68 MPa

---

**Dilation Angle (\( \gamma \))**

- At Peak Shear Strength: N/A
- Dip of Shear Surface: N/A
- Corrected Dilation Angle: N/A

---

**Bottom half post-shear**

**Top half post-shear**
B1-2-2-Fbc-8a  $\sigma_n = 8$ MPa

Shear Stiffness
Secant = 6513 MPa/m
Tangent = 6660 MPa/m
Pseudo-secant = 7600 MPa/m
Residual = 5.44 MPa

Shear Stress
Peak = 9.98 MPa

Dilation Angle ($\psi$)
At Peak Shear Strength = N/A
Dip of Shear Surface = N/A
Corrected Shear Angle = N/A

Bottom half post-shear
Top half post-shear
B1-2-3-Fbc-8b  $\sigma_n = 8\,\text{MPa}$

**Shear Stiffness**
- Secant = 5186 MPa/m
- Tangent = 5322 MPa/m
- Pseudo-secant = 5806 MPa/m
- Residual = 5.68 MPa

**Shear Strength**
- Peak = 7.76 MPa

**Dilation Angle ($\Psi$)**
- At Peak Shear Strength = N/A
- Dip of Shear Surface = N/A
- Corrected Dilation Angle = N/A

**Bottom half post-shear**

**Top half post-shear**
2 inch Diameter
Intrablock
**Shear Stiffness**
- Secant = 3053 MPa/m
- Tangent = 5324 MPa/m
- Pseudo-Secant = 5219 MPa/m

**Shear Strength**
- Peak = 3.66 MPa
- Residual = 2.12 MPa
- Ultimate = 1.39 MPa

At Peak Shear Strength = 35°
Dip of Shear Surface = N/A
Corrected Dilation Angle = N/A
$B2-2-5-B-I1-0.2b \quad \sigma_n = 0.2 \text{ MPa}$

**Shear Stiffness**
- Secant = 3412 MPa/m
- Tangent = 6462 MPa/m
- Pseudo-Secant = 5919 MPa/m

**Shear Strength**
- Peak = 3.21 MPa
- Residual = 2.44 MPa
- Ultimate = 1.78 MPa

**Dilation Angle ($\Psi$)**
- At Peak Shear Strength = $71^\circ$
- Dip of Shear Surface = N/A
- Corrected Dilation Angle = N/A

**Equations**
- $y = 9.993x + 0.106 \quad R^2 = 0.9221$
- $y = 67.569x - 2.238 \quad R^2 = 0.9230$
B1-2-2-A-11-0.5a  $\sigma_n = 0.5$ MPa

Shear Stiffness
- Secant = 3819 MPa/m
- Tangent = 6025 MPa/m
- Pseudo-Secant = 6318 MPa/m

Shear Strength
- Peak = 3.29 MPa
- Residual = 1.80 MPa
- Ultimate = 1.12 MPa

Dilation Angle ($\Psi$)
- At Peak Shear Strength = 65°
- Dip of Shear Surface = N/A
- Corrected Dilation Angle = N/A

Bottom half post-shear  Top half post-shear
**B1-2-4-C-I1-0.5b**  $\sigma_n = 0.5 \text{ MPa}$

### Shear Stiffness
- **Secant** = 3453 MPa/m
- **Tangent** = 5659 MPa/m
- **Pseudo-Secant** = 5938 MPa/m
- **Peak** = 2.76 MPa
- **Residual** = 1.50 MPa
- **Ultimate** = 0.95 MPa

### Dilation Angle ($\Psi$)
- At Peak Shear Strength = 78°
- Dip of Shear Surface = 1°
- Corrected Dilation Angle = 77°

### Normal Stress vs. Normal Displacement
- $y = 11.099x + 0.102$
- $R^2 = 0.9742$

### Normal Displacement vs. ln($\sigma_n$)
- $y = 43.471x - 2.093$
- $R^2 = 0.9596$

---

**Bottom half post-shear**

**Top half post-shear**
**B1-2-1-A-I1-2a**  \( \sigma_n = 2 \text{ MPa} \)

- **Shear Stiffness**
  - Secant = 3957 MPa/m
  - Tangent = 5168 MPa/m
  - Pseudo-Secant = 6465 MPa/m

- **Shear Strength**
  - Peak = 3.94 MPa
  - Residual = 3.77 MPa
  - Ultimate = 2.11 MPa

- **Dilation Angle** (\( \gamma \))
  - At Peak Shear Strength = 46°
  - Dip of Shear Surface = 3°
  - Corrected Dilation Angle = 43°

---

**Bottom half post-shear**

**Top half post-shear**
**B2-2-5-A-l1-2b  σₙ = 2 MPa**

**Shear Stiffness**
- Secant = 3862 MPa/m
- Tangent = 5185 MPa/m
- Pseudo-Secant = 6299 MPa/m

**Shear Strength**
- Peak = 3.62 MPa
- Residual = 3.04 MPa
- Ultimate = 2.30 MPa

**Dilation Angle (°)**
- At Peak Shear Strength = 49°
- Dip of Shear Surface = 2°
- Corrected Dilation Angle = 47°

**Graphs**
- **a:** Shear Stress (τ) vs. Shear Displacement (δₛ) (mm)
- **b:** Shear Stress (τ) vs. Shear Displacement (δₛ) (mm) with peak shear strength highlighted.
- **c:** Normal Displacement (δₙ) vs. Shear Displacement (δₛ) (mm)
- **d:** Normal Stress (σₙ) vs. Normal Displacement (δₙ) (mm) with linear fit (y = 14.865x - 0.171, R² = 0.9109).
- **e:** Natural Log of Normal Stress (ln(σₙ)) vs. Normal Displacement (δₙ) (mm) with linear fit (y = 20.612x - 1.860, R² = 0.9836).

**Images**
- Bottom half post-shear
- Top half post-shear
B2-2-5-B-13-2b \( \sigma_n = 3 \text{ MPa} \)

- Shear Strength: 7.93 MPa
- Peak = 7.93 MPa
- Residual = 7.00 MPa
- Ultimate = 4.50 MPa

- Shear Stiffness:
  - Secant = 4369 MPa/m
  - Tangent = 7839 MPa/m
  - Pseudo-Secant = 7066 MPa/m

- Dilation Angle: 53°
- Dip of Shear Surface = 4°
- Corrected Dilation Angle = 49°
B2-2-4-B-11-5a $\sigma_n = 5$ MPa

Shear Stiffness
- Secant = 5616 MPa/m
- Tangent = 8649 MPa/m
- Pseudo-Secant = 7976 MPa/m

Shear Strength
- Peak = 10.34 MPa
- Residual = 5.26 MPa
- Ultimate = 4.88 MPa

Dilation Angle ($\gamma$)
- At Peak Shear Strength = 73°
- Dip of Shear Surface = N/A
- Corrected Dilation Angle = N/A
B1-2-3-A-11-8a  $\sigma_n = 8\,\text{MPa}$

Shear Stiffness
- Secant = 6179 MPa/m
- Tangent = 8677 MPa/m
- Pseudo-Secant = 8364 MPa/m

Shear Strength
- Peak = 13.33 MPa
- Residual = 9.94 MPa
- Ultimate = 6.79 MPa

Dilation Angle ($\gamma$)
- At Peak Shear Strength = 60°
- Dip of Shear Surface = N/A
- Corrected Dilation Angle = N/A

Bottom half post-shear

Top half post-shear

Bottom half post-shear

Top half post-shear
**B2-2-5-B12-8b \( \sigma_n = 8 \text{ MPa} \)**

**Shear Stiffness**
- Secant = 6019 MPa/m
- Tangent = 7617 MPa/m
- Pseudo-Secant = 7897 MPa/m

**Shear Strength**
- Peak = 10.54 MPa
- Residual = 9.24 MPa
- Ultimate = 6.39 MPa

**Dilation Angle \( \Psi \)**
- At Peak Shear Strength = 23°
- Dip of Shear Surface = N/A
- Corrected Dilation Angle = N/A

**Bottom half post-shear**

**Top half post-shear**

**Shear Stress, \( \tau \) (MPa)**

**Shear Displacement, \( \delta_s \) (mm)**

**Normal Stress, \( \sigma_n \) (MPa)**

**Normal Displacement, \( \delta_n \) (mm)**

**\( y = 12.549x + 1.255 \)
**
- \( R^2 = 0.9700 \)**

**\( y = 3.997x + 0.229 \)
**
- \( R^2 = 0.6623 \)**
3 inch Diameter
Fracture
C2-3-2-Fab-3a  $\sigma_n = 3$ MPa

Shear Stiffness
- Secant = 2859 MPa/m
- Tangent = 3290 MPa/m
- Pseudo-secant = 3667 MPa/m
- Residual = 2.84 MPa

Shear Strength
- Peak = 4.02 MPa

Dilation Angle ($\Psi$)
- At Peak Shear Strength = 7°
- Dip of Shear Surface = 0°
- Corrected Dilation Angle = 7°

Bottom half post-shear  Top half post-shear
C2-3-13-Fbc-3b  $\sigma_n = 3$ MPa

Shear Stiffness
- Secant = 2496 MPa/m
- Tangent = 2484 MPa/m
- Pseudo-secant = 3619 MPa/m
- Residual = 2.34 MPa

Shear Strength
- Peak = 3.71 MPa

Dilation Angle ($\gamma$)
- At Peak Shear Strength = 21°
- Dip of Shear Surface = 1°
- Corrected Dilation Angle = 20°
C2-3-9-Fbc-8b \( \sigma_n = 8 \text{ MPa} \)

**Shear Stress**
- Secant = 3594 MPa/m
- Tangent = 4148 MPa/m
- Pseudo-secant = 4335 MPa/m

**Shear Strength**
- Peak = 9.85 MPa
- Residual = 5.96 MPa

**Dilation Angle (\( \psi \))**
- At Peak Shear Strength = 67°
- Dip of Shear Surface = N/A
- Corrected Dilation Angle = N/A

---

**Bottom half post-shear**

**Top half post-shear**
3 inch Diameter
Intrablock
Shear Stiffness
- Secant = 2593 MPa/m
- Tangent = 3751 MPa/m
- Pseudo-Secant = 3811 MPa/m

Shear Strength
- Peak = 4.77 MPa
- Residual = 1.23 MPa
- Ultimate = 1.03 MPa

Dilation Angle ($\psi$)
- At Peak Shear Strength = 85°
- Dip of Shear Surface = N/A
- Corrected Dilation Angle = N/A

Bottom half post-shear
Top half post-shear
$B1-3-1/l2-1.2a \quad \sigma_n = 1.2 \text{ MPa}$

Shear Stiffness
- Secant = 2374 MPa/m
- Tangent = 3786 MPa/m
- Pseudo-Secant = 3896 MPa/m

Shear Strength
- Peak = 5.36 MPa
- Residual = 4.85 MPa
- Ultimate = 3.06 MPa

Dilation Angle ($\Psi$)
- At Peak Shear Strength = 68°
- Dip of Shear Surface = N/A
- Corrected Dilation Angle = N/A

Bottom half post-shear

Top half post-shear

Bottom half post-shear

Top half post-shear
Shear Stiffness
- Secant = 2598 MPa/m
- Tangent = 3691 MPa/m
- Pseudo-Secant = 3937 MPa/m

Shear Strength
- Peak = 4.80 MPa
- Residual = 2.38 MPa
- Ultimate = 1.44 MPa

Dilation Angle ($\psi$)
- At Peak Shear Strength = 83°
- Dip of Shear Surface = N/A
- Corrected Dilation Angle = N/A

Bottom half post-shear
Top half post-shear
C2-3-B-I1-2a $\sigma_n = 2$ MPa

**Shear Stiffness**
- Secant = 2650 MPa/m
- Tangent = 3676 MPa/m
- Pseudo-Secant = 3951 MPa/m

**Shear Strength**
- Peak = 5.41 MPa
- Residual = 2.97 MPa
- Ultimate = 1.91 MPa

**Dilation Angle ($\gamma$)**
- At Peak Shear Strength = 80°
- Dip of Shear Surface = 1°
- Corrected Dilation Angle = 79°

**Bottom half post-shear**

**Top half post-shear**
**B1-3-6-L1-2b $\sigma_n = 2$ MPa**

- **Shear Stiffness**
  - Secant = 2922 MPa/m
  - Tangent = 4072 MPa/m
  - Pseudo-Secant = 4292 MPa/m
- **Shear Strength**
  - Peak = 7.09 MPa
  - Residual = 1.89 MPa
  - Ultimate = 2.26 MPa

**Dilation Angle ($\psi$)**
- At Peak Shear Strength = 80°
- Dip of Shear Surface = N/A
- Corrected Dilation Angle = N/A

**Bottom half post-shear**

**Top half post-shear**
$B1-3-1-L3-3a \quad \sigma_n = 3 \text{ MPa}$

**a**

Shear Stress, $\tau$ (MPa)

- Peak Shear Strength
- Secant = 2876 MPa/m
- Tangent = 3738 MPa/m
- Pseudo-Secant = 4435 MPa/m
- Shear Strength
- Peak = 7.20 MPa
- Residual = 5.28 MPa
- Ultimate = 3.12 MPa

**b**

Shear Displacement, $\delta_s$ (mm)

- Normal Stress, $\sigma_n$ (MPa)

- Normal Displacement, $\delta_n$ (mm)

**c**

Dilation Angle ($\psi$)

- At Peak Shear Strength = 57°
- Dip of Shear Surface = N/A
- Corrected Dilation Angle = N/A

**d**

$y = 7.994x + 0.012$

**e**

$y = 7.083x - 1.142$

Bottom half post-shear

Top half post-shear
B2-3-1-l1-8b $\sigma_n = 8$ MPa

Shear Strength
Peak = 12.84 MPa
Residual = 6.17 MPa
Ultimate = 5.96 MPa

Shear Stiffness
Secant = 3648 MPa/m
Tangent = 4641 MPa/m
Pseudo-Secant = 4678 MPa/m

Dilation Angle ($\psi$)
At Peak Shear Strength = $-57^\circ$
Dip of Shear Surface = N/A
Corrected Dilation Angle = N/A
Appendix C

X-Ray Diffraction Sample Preparation Procedure
C.1 Sample Crushing and XRD Test

The general sample preparation procedures for X-Ray Diffraction test samples are described in this Appendix. The rock hand samples were mechanically crushed in four stages to a powder for XRD testing. The first two stages of large scale crushing broke the hand sample into aggregate with grain sizes less than sand-gravel (see Figure C-1). These samples were further ground using a manual mortar and pestle (Figure C-2a). The final stage of crushing to the powder was accomplished using a micronizer, where the samples were placed in a container with quartz beads and water and vibrated in the micronizer for 5 minutes (Figure C-2b). After micronizing, the samples were placed in glass beakers and oven dried. The oven dried powdered samples were prepared for XRD testing in backpack containers (Figure C-2(d-e)) and tested using a Phillips Panalytical X’pert Pro Multi-purpose Diffractometer with an X’celerator detector (Figure C-3).

Figure C-1: Two stages of large scale mechanical crushing; (a-c) Chipmunk crusher is the first stage of crushing from hand samples to maximum gravel sized aggregate; (d-f) smaller crusher is the second stage of crushing to maximum sand-gravel sized aggregate.
Figure C-2: (a) mortar and pestle is the third stage of sample crushing to fine grained material; (b) micronizing is the fourth and final stage of mechanical crushing of the sample to very fine grained material, where the sample is placed in a container with quartz beads and water that is vibrated (in the micronizer machine at left) for 5 minutes; (c) after micronizing, the samples are placed in glass beakers in the oven at less than 80 degrees Celsius for drying; (d) dried samples are placed in backpack sample holders for XRD test; (e) completed XRD backpack samples that are ready for testing.
C.2 Hydrochloric Acid Treatment

Following the baseline XRD test of the powder samples that predominantly showed peaks representing calcite and quartz, the powder samples were treated with a 20% hydrochloric acid solution until the calcite had fully reacted, and subsequently retested with the XRD (see Figure C-4). The dry powdered sample was evenly split into two 50 ml centrifuge sample tubes. Small volumes (less than 5 ml) of hydrochloric acid (HCl) were added evenly to the sample tubes, causing an immediate fizzing reaction with the calcite particles in the sample. The HCl was carefully stirred into the dry sample until the fizzing
stopped. Additional small volumes of HCl were added to the sample tubes and stirred again until the reaction stopped. This process continued until the volume of the sample and reacted HCl liquid in the sample reached approximately 40 ml in each sample tube. At this point, the sample had reacted fully. To test this, the capped centrifuge tubes were shaken vigorously, and no reaction was observed.

To isolate the reacted sample from the remaining liquid, the sample tubes were centrifuged at 2000 RPM for 10 minutes. The remaining liquid was decanted. The sample tubes were then filled with RO water to approximately 20 ml in each tube, vortexed to suspend the sample particles, and the centrifugation was repeated with the same settings. This cleaning process was repeated three times with RO water, and the final decanted RO water was discarded. The samples were transferred to glass beakers and oven dried before subsequent XRD testing.
Figure C-4: Hydrochloric acid treatment process of XRD sample after oven drying; (a-c) crushing clumps of dry powdered sample on kimwipe; (d) pouring dry powdered sample evenly into two 50 ml centrifuge sample tubes; (e-g) adding 20% HCl solution to sample in small amounts, which fizzes with reaction to calcite in sample, following by stirring to ensure sample fully reacts; this process required filling the sample tubes in small increments (less than 5 ml at a time) to approximately 40 ml before sample had reacted fully.
C.3 Clay Separation

Following calcite treatment with HCl, the XRD results still showed dominant peaks representing quartz. Therefore, a clay separation procedure was designed to remove the larger quartz grains and isolate the smaller clay mineral grains. The clay separation was conducted using multiple stages of centrifugation (see Figure C-5). The dry powdered sample was placed in a plastic tri-corner beaker and approximately 50 ml of RO water was added to the beaker. The sample was sonicated for 60 seconds with the sonic finger to suspend the sample particles in the RO water. Four glass beakers were labelled with the final fraction sizes: >10, 5-10, 2-5, and <2 microns (μm). The sonicated sample was evenly separated into two 50 ml centrifuge sample tubes and placed in opposite sides of the centrifuge container for weight balance.

To extract the first, >10 μm, fraction, the sample was spun in the centrifuge for 30 seconds at a speed of 500 RPM. The liquid portions of the separated samples, where sample size fractions <10 μm were suspended in the water, were decanted into a clean pair of centrifuge tubes. The compacted sample that remained in the tube after decanting represents the >10 μm sample size fraction. Approximately 10 ml of RO water was added to the tubes containing the compacted samples and the samples were vibrated with a vortex machine and then poured into the prepared glass beaker for the >10 μm size fraction. The remaining fractions were separated with the following centrifuge settings and correspondingly placed in each labelled glass beaker:

- 5-10 μm fraction: 30 seconds at 1000 RPM
- 2-5 μm fraction: 30 seconds at 2000 RPM
- <2 μm fraction: 60 seconds at 8000 RPM

These centrifugation settings were recommended by Dr. Steve Beyer, following his application of the procedures described by Gimbert et al. (2005). The size fractionated samples were oven dried for subsequent XRD testing.
Figure C-5: Clay separation procedure; (a) sonic finger used to suspend sample in RO water; (b-d) sonicated sample poured evenly into two 50 ml centrifuge tubes and sealed with threaded caps; (e-f) samples placed in opposite sides of centrifuge container for weight balance and secured inside with watertight seal; (g) sample is separated from water after centrifugation; (h) several stages of centrifugation resulted in approximate bins of sample by grain size (<2, 2-5, 5-10, and >10 μm).
C.4 References

Appendix D

MLA Mineral Reference Library for the Cobourg Limestone
Quartz

Full Spectrum

100 Channel = 1.00 keV

Counts

Channel

Zoom

Counts

Channel

Si
(Ka1 = 174)

O
(Ka1 = 52)
Ankerite

Full Spectrum

100 Channel = 1.00 keV

Counts

Channel

Ca (Ka1 = 369)

Mg (Ka1 = 125)

C (Ka1 = 28)

O (Ka1 = 52)

Ca (Kb1 = 401)

Fe (Ka1 = 640)

Counts

Channel

D-4
Biotite

Full Spectrum

100 Channel = 1.00 keV

Counts

Channel

Si
(Kα1 = 174)

Al
(Kα1 = 149)

O
(Kα1 = 52)

Mg
(Kα1 = 125)

K
(Kα1 = 331)

Ca
(Kα1 = 369)

Fe
(Kα1 = 640)

Counts

Channel

D-6
Glauconite

Full Spectrum

100 Channel = 1.00 keV

Counts

Channel

Si
(Ka1 = 174)

Zoom

Counts

Channel

O
(Ka1 = 52)

Al
(Ka1 = 149)

Mg
(Ka1 = 125)

K
(Ka1 = 331)

Ca
(Ka1 = 369)

Ca
(Kb1 = 401)

Fe
(Ka1 = 640)

Fe
(Kb1 = 706)
Albite

Full Spectrum
100 Channel = 1.00 keV

Counts
Channel

Zoom

Si
(Ka1 = 174)

O
(Ka1 = 52)

Al
(Ka1 = 149)

Na
(Ka1 = 104)

Fe
(Ka1 = 640)

Counts
Channel
Pyrite

Full Spectrum

100 Channel = 1.00 keV

Counts

Channel

S
(Ka1 = 231)

Fe
(Ka1 = 640)

Fe
(La1 = 70)

Fe
(Kb1 = 706)

Counts

Channel

D-9
Gypsum

**Full Spectrum**

100 Channel = 1.00 keV

**Counts**

- **O** (Ka1 = 52)
- **S** (Ka1 = 231)
- **Ca** (Ka1 = 369)
- **O** (Ka1 = 52)
- **Ca** (Kb1 = 401)
Apatite

Full Spectrum

100 Channel = 1.00 keV

Counts

Channel

Ca (Ka1 = 369)

Zoom

Counts

Channel

P (Ka1 = 201)

Ca (Kb1 = 401)

O (Ka1 = 52)
Sphalerite

Counts

Full Spectrum

100 Channel = 1.00 keV

Counts

Na (Ka1 = 104)

S (Ka1 = 231)

Zn (Ka1 = 864)

Zn (Kb1 = 957)