STRESS ESTIMATION FROM BOREHOLE SCANS FOR PREDICTION OF EXCAVATION OVERBREAK IN BRITTLE ROCK

by

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Abstract

In the field of geomechanics, one of the most important considerations during design is the state of stress that exists at the location of a project. Despite this, stress is often the most poorly understood site characteristic, given the current challenges in accurately measuring it. This stems from the fact that stress can’t be directly measured, but must be inferred by disturbing the rockmass and recording its response. Although some methods do exist for the prediction of in situ stress, this only provides a point estimate and is often plagued with uncertain results and practical limitations in the field. This research proposes a methodology of continuously predicting stress along a borehole through the back analysis of borehole breakout and how this same approach could be employed to predict excavation overbreak.

KGHM’s Victoria Project in Sudbury, Canada, was the location of data collection, which firstly involved site characterization through common geotechnical core logging procedures and laboratory scale intact core testing. Testing comprised Brazilian tensile strength and unconfined compressive strength testing, which involved the characterization of crack accumulation in both cases.

From two pilot holes, acoustic televiewer surveys were completed to characterize the occurrence and geometry of breakout. This was done to predict the orientation of major principal stresses in the horizontal axis, with the results being further validated by the geometry of stress-induced core disking. From the lab material properties and breakout geometries, a continuum based, back analysis of breakout was done through the creation of a generic database of stress dependent numerical models. When compared with the in situ breakout profiles, this created an estimate of stress as a function of depth along each hole. The consideration of the presence of borehole fluid on the estimate of stress was also made. This provided the upper-bound estimate of stress from this methodology. Given the generic nature of the numerical models, potential shaft overbreak was also assessed using this technique and from the previously described estimate of stress.
Co-Authorship

The thesis “Development of an In Situ Stress Estimation Methodology from Borehole Breakout for the Prediction of Excavation Overbreak in Brittle Rock” is the product of research completed by the author, Andrew LeRiche. Throughout, scientific and editorial feedback has been provided by Dr. Mark Diederichs and Dr. Kathy Kalenchuk, however the written content is solely that of Andrew LeRiche. Any contributions made by these or other colleagues in the form of conference papers, have been acknowledged in the contributions and references section.
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<td>AE</td>
<td>Acoustic Emission</td>
</tr>
<tr>
<td>AECL</td>
<td>Atomic Energy of Canada</td>
</tr>
<tr>
<td>ASTM</td>
<td>American Section of the International Association for Testing Materials</td>
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<td>ATV</td>
<td>Acoustic Televiewer</td>
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<td>BEM</td>
<td>Boundary Element Method</td>
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<td>BOA</td>
<td>Breakout Opening Angle</td>
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<td>BTS</td>
<td>Brazilian Tensile Strength</td>
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<td>BWS</td>
<td>Borehole Wall Strength</td>
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<td>c</td>
<td>cohesive strength</td>
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<td>Crack Closure</td>
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<td>CD-DAS</td>
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<td>CI</td>
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<td>CWFS</td>
<td>Cohesion Weakening Friction Strengthening</td>
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<td>DEM</td>
<td>Discrete Element Model</td>
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<td>Discrete Fracture Network</td>
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<td>$FF$</td>
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<td>HTPF</td>
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<td>ISRM</td>
<td>International Society for Rock Mechanics</td>
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<td>ITLS</td>
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<td>LVDT</td>
<td>Linear Variable Differential Transformer</td>
</tr>
<tr>
<td>$\text{mbgs}$</td>
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<td>National Instrument for Mineral Project Reporting in Canada</td>
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<tr>
<td>PFC</td>
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<td>$K_{xy}$</td>
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<td>$P_b$</td>
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Chapter 1

Introduction

1.1 Significance of In Situ Stress Characterization

During deep tunneling or mine infrastructure development, the assumed stress state has significant implications on geomechanical design. As the mineral extraction industry looks to develop deeper and more technically demanding deposits, properly characterizing the state of stress has an increasingly important role in this process. In such high risk settings, the potential magnitude of overbreak or ore dilution can have repercussions on an entire mine’s life, from feasibility studies and mine design to closure and rehabilitation. Properly constraining both the orientation and magnitude of stress at earlier stages in a project can allow for greater levels of certainty in key design inputs, which may ultimately influence a project’s viability.

In geomechanics, the selection and implementation of a successful design is often due in large part to the adequate collection and analysis of available data. In many fields of underground construction, the chosen method of design is often empirical, which allows for the simplification of many inputs. Although this understandably reduces the quantity and complexity of data which must be collected, it can sometimes lead to shortcomings in the ultimate design. To account for this uncertainty, designs often become highly conservative or are mistakenly chosen based on an empirical system which was not created and calibrated for the specific ground conditions in question.

Although the use of numerical modelling has been well documented since the 1980’s (Hoek and Brown, 1980; Cundall, 1988), it was largely regarded as an academic exercise until more recent developments in both computing power and understanding of complex rockmass behaviour. The collected in situ data is required for the definition of key design factors, such as stress and displacement around an excavation, while also verifying the validity of the chosen support and reinforcement systems. One of the
most powerful uses of numerical modelling is to back analyze observed behaviour of the rockmass once the design has been implemented. This not only helps to validate the assumptions used in the model but can also be used to improve modelling confidence for other excavations in the same rockmass. As has been shown in the past, with adequate data, the back analysis of excavation scale deformation can be used to gain insight into the local stress state (Jaeger and Cook, 1976). The body of work contained herein also aims to develop a similar methodology of numerical back analysis of borehole scale deformation to not only quantify the local stress state but also predict potential magnitudes of excavation overbreak. This is in the motivation of creating another numerical tool for use in practical applications of rock engineering.

1.1.1 Challenges of In Situ Stress Estimation

Despite the importance which stress has as an engineering input, to date, remote measurement of the three-dimensional stress state at depth has proven to be a significant challenge. In many cases, stress is assumed from either regional historic tests or based on the tectonic setting. This can be attributed to two predominant reasons- 1) The cost and consequent inadequate amount of stress data collected and 2) the existing uncertainty with the currently accepted measuring techniques.

In the case of a mining project at the scoping to feasibility level, the estimation of stress through measurement is often dictated by the use of deep borehole investigation techniques. This is due to the fact that at this point in the project, no underground infrastructure exists to aid in the back analysis of stress. Using these remote techniques results in both logistical and technical shortcomings which creates difficulty in estimating stress with any degree of confidence. Whether it is by borehole overcoring, hydraulic fracturing of the borehole or the use of borehole breakout, they all presume that the exact conditions at the measurement location are understood. Even if this assumption holds true, it has been shown that in a perfectly elastic and isotropic rockmass the measured stress orientation and magnitude can vary from 10-25% of the actual stress state (Amadei and Stephansson, 1997). This is without considering the addition of factors which are often present at the borehole scale, such as rockmass heterogeneity or borehole fluid.
To quantify the spatial variability in rockmass properties, lab testing at regular intervals along the length of the borehole must be done, which represents another point of data collection where error or uncertainty can be introduced into the ultimate stress estimation. Using such borehole methodologies, it is often suggested to implement multiple methods to attempt to reduce the degree of uncertainty with each individual measurement. However, with this idealized approach it is assumed that tests can be frequently done along the length of the borehole. For the case of overcoring or hydro-fracturing, this not only presents a logistical issue but also a financial one. This is where, if a strong understanding of rockmass conditions as a function of depth exists, the use of surveying tools such as an acoustic televiewer (ATV) can be used to get a continuous idea of deformation along the length of the borehole. As shown in this body of work, this allows for the prediction of stress continuously along the borehole, provided the proper assumptions are made.

From the currently proposed framework of stress estimation (Figure 1-1), the back analysis of borehole breakout is considered a stress measurement method, although it isn’t included within the integration of stress results from other methods (step 3). With the proposed methodology in this dissertation, the idea is that borehole deformation can be used as another tool when assessing stress. This not only presents a more spatially continuous image of stress, but one that is much more cost effective when the quantity of data is considered as compared to more conventional stress estimation techniques.
1.1.2 Stress and Engineering Design

Following the creation of any sort of underground opening, a natural redistribution of stress occurs around its entire periphery. If the strength of the excavation is exceeded by this concentration of stress, it causes irreversible damage to the rockmass at varying degrees, which is dependent on not just the intact strength of the rock but also the presence of heterogeneity, such as joint networks. If great enough, this post-mining stress change can result in overstressed conditions, which leads to the failure of material and a further redistribution of stresses. This process and the formation of the characteristic “v-shaped” notch in brittle rock is shown below in Figure 1-2, where the occurrence of the notch occurs perpendicular to the
location of the Major Principal Stress axis, in the zone of highest post-excision stress. If the ratio of stress is sufficiently high, this zone of failure becomes pronounced with the potential for zones of tension to be formed at 90° to the notch, causing separate concerns for excavation stability. If, however the stress ratio is closer to unity, this causes uncertainty in the prediction of the location and extent of failure.

Figure 1-2 – The Mine-by-Experiment excavation at the Underground Research Laboratory in Pinawa, Manitoba. a) Shows the final geometry of the circular excavation with the v-shaped notch in the floor and back, b) a cross section through the floor of the excavation showing the notch shape, and c) highlighting the notch tip (Read, 2004).

In a sparsely jointed, brittle rockmass, it is critical to understand the in situ stress conditions prior to excavation to predict not only how the physical geometry of the opening will change as a result of the induced stress, but also the support measures that will be needed. This is dependent on the consequences of overstressing on the infrastructure. For example, in the case of permanent mining openings such as shafts and maintenance bays, the results of such failures are of more concern than in the actively
producing stopes, which is reflected in the chosen support methods. Overbreak in these environments can also have a greater financial impact, as larger volumes of rock must be moved during construction. That being said, despite the stability of the actively producing areas being less of a concern to personnel, given the lower exposure and shorter operational lifespan, it does still have the potential to greatly influence the economics of a mining operation. This is through the introduction of waste rock from overbreak material, known as dilution, into the ore feed (Figure 1-3). An example of the potential volume of dilution that can be introduced is illustrated in Figure 1-3b, as highlighted by the blue volumes. This inevitable occurrence of dilution is important to characterize at all stages of due diligence when planning a mining project, with the pre-mining stress state being a key input to the analysis.

![Figure 1-3 - Example of sublevel mining operation with a) typical sequencing and extraction points (after Atlas Copco 2007) and b) predicted levels of stope overbreak in blue which results in dilution of economic ore (after Stephenson & Sandy 2013).](image)

The process of designing an underground excavation is one which has many inputs. The progression to a final design starts with three main considerations; mechanical parameters of the rockmass, stress state and importance/permanence of the infrastructure (Figure 1-4). It is clear that not
properly quantifying one of these initial inputs has a trickle down effect all the way to the implementation of design. This is why, in high risk mining or civil infrastructure developments, having a detailed understanding of the stress conditions allows for more informed decisions to be made throughout all aspects of the design. This process is also highly iterative in nature, leading to a continued increase in knowledge as design and construction progresses and more data collection techniques can be employed.

![Diagram of the design process of underground excavations](image)

Figure 1-4 – Simplified process of the design of underground excavations (Martin, Kaiser and Christiansson, 2003).

### 1.2 KGHM- Victoria Project

The site of data collection for the completed work was at KGHM’s Victoria Project, along the western margin of the Sudbury Basin’s South Range (Figure 1-5). Mining of near surface Ni-Cu-PGE mineralization began in 1899 under the ownership of The Mond Nickel Company which was subsequently purchased by the International Nickel Company (Inco.) in 1929. Between 1970 and 1978 the property saw further mining until eventual closure of the near surface mine. In 2002 the property was optioned by The FNX Mining Company from Inco, and exploration for deeper economic mineralization began in 2008. In 2010, Quadra FNX announced the discovery of a high-grade zone at depth (Zone 4), with an inferred resource estimate announced in 2011. It was in the same year that Quadra FNX was acquired by KGHM International, a state owned Polish mining company. Project development began in 2013 with the construction of surface infrastructure and planned sinking of an initial exploration shaft in
2014 to prepare for eventual production in 2019. From the most recent estimate, 13.6 M tonnes of inferred resources occurs persistently format a depth of 1175-1950 meters below ground surface (mbgs). Given the steeply dipping nature of mineralization and its continuous nature, the current mine model is considering using transverse longhole stoping as the primary mining method (Farrow et al., 2011; KGHM, 2013).

As a result of low global commodity prices, the project timeline was slowed in early 2016, which remains its current situation. Despite this, large amounts of geomechanical data collection and site characterization has been done, which comprises the body of data used for the prediction of in situ stress and subsequent estimation of shaft overbreak magnitudes in this thesis.

Figure 1-5 – Map of geological provinces of North America, with the location of KGHM’s Victoria Project highlighted (after Barton et al. 2003).
1.2.1 Summary of Pilot Hole Data

For characterization of the rockmass and subsequent design of the shaft and associated infrastructure, two vertical pilot holes: FNX-1204 in 2011 and GT0020VCa in 2016, were completed. The use of two separate boreholes was dictated by the fact that shaft alignment changed with a mine plan update in 2015, which resulted in the need for the second borehole.

For FNX-1204, Golder Associates conducted data collection through core logging, down-hole ATV surveys and lab testing of recovered core samples (Corkery and Palmer, 2011). Strength testing of samples from FNX-1204 was completed by the Robert M. Buchan Department of Mining testing lab, at Queen’s University. From these results, MDEng., a mining geomechanics consulting company completed an independent assessment of the collected data for use in their geomechanical mine design (Kalenchuk and Hume, 2015). The key outcomes of that study include:

1) Critical Infrastructure siting
2) Ground support design
3) Stope sizing
4) Mining method and sequence design- to manage high stress scenarios
5) Assessment of shaft wall stability

The more recent pilot hole (GT0020VCa) was logged by SRK Consulting, with the same geomechanical properties being considered as the previous pilot hole (Zhang, 2016). Sample testing has been completed at KGHM’s labs in Poland with some testing done as part of this thesis at the Queen’s University Geomechanics Rock Testing Lab. Results from the testing in Poland and an update to the geomechanics mine design have yet to be formalized. The large majority of work contained within this thesis was used the results from the more recent hole, as it more closely aligns with the current location of proposed shaft infrastructure. A complete summary of data for both pilot holes can be found in Chapter 3.
1.3 Thesis Objectives

The work in this thesis aims to provide a methodology framework for the prediction of in situ stress and excavation overbreak using borehole scale failure and a proper geomechanical understanding of the rockmass. The objectives are sequentially ordered as follows:

- Critically review current stress estimation practices to identify their relative strengths/shortcomings with particular relation to early stage mining projects.
- Assess the intact properties of various lithologies intersected along the length of the proposed shafts, through lab-scale testing of unconfined compressive strength (UCS), Brazilian tensile strength (BTS) and stiffness.
- Review and interpretation of geotechnical logging data from both pilot holes, for creation of independent evaluation of rockmass properties as a function of depth along the shaft.
- Creation of a repeatable methodology for the interpretation of borehole breakout logs, such that boreholes of different diameters or containing differing frequency and size of breakout can be assessed in parallel.
- Explore the use of continuum based numerical modelling for the creation of a generalized breakout database to be used for the prediction of in situ stress magnitudes, as applied to breakout profiles from both pilot holes.
- Identify key indicators of excavation scale damage and how they can be scaled from the borehole to the excavation. Major assumptions on scalability of strength were also discussed and evaluated.
- Assess the applicability of using the same breakout database for the prediction of various excavation damage zones. This is demonstrated by taking the same breakout profiles and estimated stress magnitudes for reinterpretation of excavation overbreak.
- Delineate the sensitivity of the in situ stress/excavation overbreak results based on the model input assumptions.
Discuss the applicability of the proposed methodology and where current drawbacks still remain.

This is to highlight future potential work on this project once mining development begins.

1.4 Thesis Outline

This thesis has been written as per the guidelines and requirements set out by the School of Graduate Studies at Queen’s University, Kingston, Ontario. The thesis is written in a traditional format and consists of eight chapters which are outlined as follows:

Chapter 1: Discusses, at a high level, the concepts of in situ stress and excavation scale damage with emphasis on some of the current challenges in engineering associated with these. The importance of properly characterizing stress for underground infrastructure is highlighted with particular attention being paid to the mining industry. Chapter 1 also brings into context the site of research for this dissertation at KGHM’s Victoria Project in Sudbury, Ontario.

Chapter 2: Highlights the background information necessary to this research, which includes in situ stress concepts, the stress-related mechanisms which lead to excavation overbreak, the use of numerical modeling and the tectonic evolution of the Sudbury basin with implications on the local geology surrounding the Victoria Project.

Chapter 3: Details the results from all forms of data collection from both pilot holes FNX-1204 and GT0020VCa. This includes a general description of all lithologies encountered from a geological and geomechanical perspective, their structural interpretation and laboratory testing which was done in the past and as part of this thesis.

Chapter 4: Demonstrates a methodology for the interpretation of borehole breakout from the use of acoustic televiewer data. This is applied to both pilot holes, where the breakout profiles are quantified and the orientation of stress is reported. This is cross validated with the log of disking orientation taken from oriented core from borehole GT0020VCa.

Chapter 5: The approach of numerical modelling for the prediction of in situ stress and excavation overbreak magnitudes is described with particular focus on the challenges associated with
stress evaluation, the constitutive model, material property sensitivity and the procedure of interpreting modeling results.

Chapter 6: The procedure of incorporating the in situ breakout profiles with modelling results for the prediction of as a function of depth in both pilot holes. This produces the estimated stress state along the shaft alignment at the Victoria Project. Attention is given in this chapter to the importance of effective strength assumptions necessary at the borehole/excavation scale. Sensitivity of the results are explored as they relate to the presence of borehole fluid pressure and material property variance with depth.

Chapter 7: Implementation of the proposed stress estimation into a practical framework is discussed. Various scenarios are considered which include; the use of multiple boreholes for the creation of a 3D stress model, the influence of material property assumptions on stress prediction, the upscaling of results to excavation overbreak and future considerations of the methodology.

Chapter 8: Provides a discussion of the applicability of results to practical engineering applications with key limitations. Ideas for future work to further develop this methodology is explored, with concluding remarks on this body of work.
Chapter 2

Literature Review

2.1 Development of the Sudbury Basin

One of the most prolific mining districts in the world is found surrounding the Sudbury Basin within the Canadian Shield. The creation of Ni-Cu-PGE deposits is associated with one of the largest known terrestrial meteor impact structures, which created what is formally known as the Sudbury Igneous Complex (SIC)- (Long, 2004). This roughly elliptical basin is represented by the post impact sedimentary rocks of the Whitewater Group, the impact related melt breccias of the Onaping Formation, the synformal SIC and the underlying country rock of the Superior and Southern Provinces. The SIC is divided into the North, South and East Ranges which show distinct characteristics such as melt thickness, lithological contact dips and structural overprinting (Figure 2-1). These features have largely been attributed to pre and post tectonic activity, with higher degrees of deformation seen along the South Range Shear Zone (SRSZ)- (Lenauer, 2012). It is within the South Range where KGHM’s Victoria Project is found.

![Figure 2-1 – Plan view of Sudbury Igneous Complex with overlying sedimentary rocks, highlighting major fold and fault locations with associated mining operations (Lenauer, 2012).](image-url)
2.1.1 Tectonic Evolution

Broadly speaking, the evolution of the Sudbury Basin can be divided into three distinct times; 1) pre-impact, 2) syn-impact and 3) post-impact. Each of these produced distinct structural fabrics and regional scale structures which not only affect mineralization but also the regional stress state in the Basin. On a local scale, the orientation of the stress tensor is quite variable and is often intimately related to the orientation of local faulting and shear zones (Cowan, Riller and Schwerdtner, 1999; Santimano and Riller, 2012). It is because of this that it is important to understand the tectonic evolution of the region to describe why the interpreted virgin stress from measurement may have a given orientation and magnitude.

2.1.1.1 Pre-Impact- Formation of the Southern Province

In contact with the Southern Range of the SIC is the Metavolcanic and Metasediment dominated Huronian Supergroup of the Southern Province. This interlayered sequence was deposited primarily in a rift basin overlying the Archean basement beginning at 2.45 Ga. As rifting subsided, volcanics and sediments continued to be deposited on the margin. Renewed mantle plume activity is thought to have caused moderate uplift and tilting (northwards) of the Huronian Supergroup in an extensional setting which created a number of regional scale East-striking normal faults such as the Murray and Creighton Fault system (Long, 2004). Deformation of the Huronian rocks occurred during the Penokean Orogeny which is associated with an island arc collision along the southern fringe of the Superior Province (Cowan, Riller and Schwerdtner, 1999). This led to a probable reactivation of the normal faults in a thrusting sense and the formation of upright and steeply dipping folds with an E/W fold axis (Long, 2004). The orogeny began just before the emplacement of the Sudbury Igneous Complex (1.85 Ga) and continued throughout this time.

2.1.1.2 Syn-Impact Related Structures

The result of the meteorite impact was the creation of an impact melt sheet and crater up to 3 km deep (Lenauer, 2012). This regionally created the melt related breccias, shatter cones and psuedotachylyte bodies which are characteristic of this area. The kinetic energy during the collision resulted in large
displacements of rock and the formation of a cavity which led to subsequent isostatic rebound and central uplift. The melt sheet is composed of mafic-ultramafic rocks which have been differentiated into distinct layers (Riller, 2005). This melt sheet also hosts brecciated rocks from comminution of the footwall rock surrounding the impact. It is along the base of the SIC, known as the sublayer, where the melt interacted with existing basement structures to create the radial quartz diorite offset dykes. It is along the sublayer and within the offset dykes where most mineralization is found in the Basin (Cowan, Riller and Schwerdtner, 1999; Riller, 2005). It is believed that the original geometry of the melt sheet was sub-horizontal to funnel shaped. As a result, significant post-emplacement deformation must have occurred to create the current geometry of the Sudbury Basin (Lenauer, 2012; Santimano and Riller, 2012). From structural relationships observed between relic structures, it is thought that the stress was compressional at this time with the Major Principal Stress in approximately the Northwest-Southeast direction.

2.1.1.3 Post-Impact to Present
Following the impact event, the Penokean Orogeny remained the driving force of deformation in the region (Cowan, Riller and Schwerdtner, 1999; Lenauer, 2012). At this time the Sudbury region was situated in the foreland basin, which led to the deposition of the Whitewater Group at the center of the SIC. The later stage of this orogeny is associated with brittle deformation, creating the most notable feature being the South Range Shear Zone, which transects much of the Basin (Figure 2-1). This also led to the re-activation of previously occurring structures such as the regional scale Murray and Creighton Faults. As can be seen in Figure 2-2, this compressional event also caused back thrusting to occur in the South Range, which gives the Basin it’s apparent sense of being overturned. Between 1.2-1.1 Ga, the Grenville Orogeny occurred, causing shortening of the Sudbury Structure, and further reactivation of existing faults (Long, 2004; Santimano and Riller, 2012). It was the deformation during this time and during the Penokean Orogeny which is responsible for the overturned, doubly-plunging synformal nature of the Basin (Mungall et al., 2004). As a result of the crustal thinning and mantle upwelling caused by the impact, isostatic rebound has forced the down-cutting and erosion of much of the region. This has
removed most of the original SIC to its current level (Lenauer, 2012). Multiple glacial events since the meteorite impact have also caused further erosion and crustal loading in the Basin (Dyke et al., 2002).

Figure 2-2 – Cross section of the Eastern portion of the Sudbury Basin, showing post-impact related structural features (Cowan, Riller and Schwerdtner, 1999).

2.2 Geology of the Victoria Deposit
The Ni-Cu-PGE brownfield development at The Victoria Deposit is associated with mineralization in the Worthington Offset Dyke, which is proximal to the regional scale Creighton Fault Structure (Figure 2-3). Along this offshoot from the main basin, other mineralized zones include the Totten Deposit, The McIntyre property and the AER/Kidd Copper property. The footwall assemblage at the location of the Victoria Deposit is comprised of Paleoproterozoic Huronian Supergroup assemblage of interbedded metavolcanics and metasediments with occasional mafic intrusions (Farrow et al., 2011). The structural trend of these units is east-southeast and steeply dipping (75° to 80°) to the south-southwest. Proximal to the offset dyke is also the interpreted location of closure and fold hinge of the synformal Sudbury Basin.
As a result, there are zones of NE/SW shearing and some variability in structural fabric orientation. Where the offset intersects the Creighton Fault, there is a dextral shift of the unit to the west. Potentially economic mineralization and the proposed location of the ventilation and production shafts are south of this fault.

Figure 2-3 – Structural map of the Southern Range of the Sudbury Igneous Complex. Inset in the top left corner is the entire Sudbury Basin with outline of study location in the South West. The Victoria Project is highlighted in the bottom left corner (after Riller 2005).

2.2.1 Planned Development

From delineation drilling of the orebody, it has been found that mineralization in the form of 13.6 M tonnes of inferred resources (2.5% Cu, 2.5% Ni, 8.3 g/ton TPM) exists from approximately 1175-1950 mbgs (Farrow et al., 2011). As can be seen in Figure 2-4, mineralization is interpreted to be continuous with depth and varies from 125-200 m in length along-strike and 25-50 m in thickness. The ore body strikes at 080° and is steeply dipping at 70° to the south east. Given the size, continuity and geometry of the ore body, the conceptual mining model is considering the use of transverse longhole stoping. The chosen stoping dimensions are a maximum panel length of 30 m, 15 m width and standard 25 m height between each level (Farrow et al., 2011). Prior to the commencement of the secondary and
tertiary stopes, the previous stope void will be backfilled using pastefill. The location and design of support infrastructure such as level access ramps and shaft development are being done by considering the induced stress from the production areas. This, along with the design of support requirements in such cases, further supports the strong consideration and investment into assessing the virgin stress state prior to the commencement of development.

The relative geometry and location of underground infrastructure development can be seen in Figure 2-4. As per the current mine plan scenario, both ventilation and productions shafts are to be sunk to 1850 mbgs. It is approximately along the axis of both shafts where two geotechnical pilot holes were used for data collection and interpretation of rockmass conditions.

Figure 2-4 – Victoria ore body with critical mining infrastructure looking in the a) approximately East direction along strike and, b) North, down dip of the ore body (KGHM, 2013).

2.3 Introduction to Stress Concepts

In physical space there exist quantities which can be defined by a number of differing variables. In the simplest case, a scalar quantity is defined by one dimension in space and time and can include things
such as hydrostatic pressure or temperature. When measurements must be defined by two variables such as direction and magnitude these are vectors, with examples including force and velocity. In the case of stress it is not only defined by magnitude and orientation but also the plane that the stress is acting upon.

The notion of stress originates from the study of the failure of solids, where the stress field, being the distribution of internal tractions, must balance with a set of external tractions and body forces. As a third-order quantity, this is known as a tensor. As can be seen in Figure 2-5, to completely describe how stress acts, nine components must be stated; three normal components (σ) and six shear components (τ). From matrix symmetry, only three shear components are needed, resulting in six parts to resolve the entire tensor.

![Diagram of stress components](image)

**Figure 2-5 – Representation of the components of stress in 3-D (left) and matrix (right) which defines all nine components of the tensor (after Hudson et al. 2003).**

The applied normal stress changes the volume of the material which is resisted by its bulk modulus, as defined by the Young’s Modulus and Poisson’s ratio. On the other hand, the shear stresses tend to deform the material without a change in volume, with resistance to this change coming from the material’s shear modulus (Amadei and Stephansson, 1997). The matrix presented in Figure 2-5 can also be resolved such that all shear components of stress are zero. This leaves the orthogonal stress components to define the stress field, which are known as the Principal Stresses. These are represented as σ₁ as the Maximum, σ₂ as the Intermediate and σ₃ as the Minor Principal Stress. For practical engineering applications, the in situ state of stress is almost always defined using its Principal Stress components.
2.3.1 Anderson’s Theory of Faulting

During engineering design, it is generally assumed that one principal component of the stress tensor is in the vertical direction ($\sigma_v$). This leaves the other two principal stresses being in the horizontal plane, that are denoted as the maximum horizontal stress ($\sigma_H$) and minimum horizontal stress ($\sigma_h$), which will be the convention used for the remainder of the dissertation. In this context, Anderson (1951) defined stress regimes and the occurrence of faulting based on $\sigma_v$ and what principal stress it is related to. The three simplified scenarios in Figure 2-6 include:

1) The maximum principal stress is oriented in the vertical direction ($\sigma_v = \sigma_1$), meaning the geological setting is in a normal faulting regime.

2) The intermediate principal stress is oriented in the vertical direction ($\sigma_v = \sigma_2$), meaning the geological setting is in a strike-slip fault regime.

3) The minimum principal stress is oriented in the vertical direction ($\sigma_v = \sigma_3$), meaning the geological setting is in a reverse or thrust faulting regime.

![Figure 2-6 – Illustration of the influence of principal stress orientations on the faulting regime, based on Anderson’s theory (after School of Earth Atmospheric and Environmental Sciences 2013).](image)

A normal faulting regime is indicative of an extensional tectonic setting which can include, but is not limited to, divergent plate boundaries or back-arc basins such as the basin and range province of the United States that show characteristic horst and graben structures. When the vertical stress is equal to $\sigma_2$,
this typically occurs in regions proximal to translating plate boundaries which are neither converging nor extensional in nature. This translational movement leads to the occurrence of strike slip faulting and associated features such as pull apart basins and push up zones. These accessory structures are evident based on the occurrence of normal faulting and reverse faulting respectively, leading to some complexity as all three stress regimes appear to occur locally within the same setting. When a stress field is said to be in a reverse faulting regime, this is indicative of a compressional setting such as a subduction zone, orogenic belt and throughout most old continental terrains. This is the case throughout much of the Canadian Shield, which comprises the study area in the Sudbury Basin (Herget, 1987).

### 2.3.2 Origins of Crustal Stress

The stress in rock at a point of interest can be divided into two domains of in situ and induced stress. This generally takes the following form:

\[
\sigma_{\text{Total}} = f(\sigma_{\text{in situ}}, \sigma_{\text{induced}})
\]

(2-1)

where \(\sigma_{\text{in situ}}\) is the primitive or virgin stress which exists in the crust and \(\sigma_{\text{induced}}\) is the stress that is caused by a disturbance to the rockmass from excavation, also known as the post-mining stress (Amadei and Stephansson, 1997).

At any location within the earth’s upper lithosphere, the stress state prior to disturbance is the sum of the regional forces acting at that location. This is largely dominated by overburden pressure and the tectonic evolution of the area (first order) but also sees local changes in magnitude and orientation from second-order features (Zoback, 1992). In a passive, mid-continental setting such as Sudbury, these local/regional variations in stress can be a result of crustal isostatic rebound from glacial melting, or contrasts in material properties in both a lateral and vertical sense. Fejerskov and Lindholm (2000) summarized the scales at which these stress-causing mechanisms act in Table 2-1.
Table 2-1 – Overview of stress generating mechanisms and the extent on which they act (after Fejerskov & Lindholm 2000).

<table>
<thead>
<tr>
<th>Stress Field- Extent</th>
<th>Continental (&gt;1000 km)</th>
<th>Regional (100-1000 km)</th>
<th>Local (&lt;100 km)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stress generating mechanisms</td>
<td>Plate Tectonic forces • Ridge push • Slab pull • Basal drag • Trench suction</td>
<td>Large-scale density inhomogeneities • Continental margin</td>
<td>Topography • Fjords • Mountain Ranges</td>
</tr>
<tr>
<td></td>
<td>Gravitational Loading</td>
<td>Flexural stresses • Deglaciation • Sediment loading</td>
<td>Geological features • Faults • Hard/soft inclusions • Folding</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Wide topographic loading</td>
<td></td>
</tr>
</tbody>
</table>

Amadei and Stephansson (1997) proposed a terminology framework to describe rock stress (Figure 2-7), which is based on the work done by a number of authors (Bielenstein & Barron 1971, Hyett et al. 1986 and Price & Cosgrove 1990). This creates a division between the naturally occurring stress and the superimposed excavation stresses as described in Eq.(2-1), with the in situ stress then being simplified into four main fields; Gravitational, Tectonic, Residual and Terrestrial.
2.3.2.1 Gravitational Loading

The weight of material located above the point of interest is what represents the component of stress caused by gravity. This directly influences the total state of stress at all scales within the crust and in the absence of any topographical features, is directly proportional to the thickness of overlying material. This can be mathematically represented as the integration of rock densities from surface to the depth of interest, and takes the generalized form of:

$$\sigma_v = \int_0^z \rho(z) g \cdot dz = \bar{\rho}g z$$  \hspace{1cm} (2-2)
Where: $\rho(z)$ = density as a function of depth

$g$ = gravitational field constant

$\bar{\rho}$ = mean overburden density

One underlying assumption of using this approach is that the surface of the earth is perfectly flat. If this is accepted, and the average density of rock is used (2.6 g/cm$^3$) then this produces a lithostatic stress gradient of 0.026 MPa/m. Although this assumption is adequate for estimating vertical stress at depth, near surface vertical stresses tend to be adversely affected by the presence of topographic relief. This is well illustrated in Figure 2-8, where it is evident that at depth, $\sigma_1$ is oriented vertically, however near the surface, it rotates to being sub-parallel to the topographic features. As a result of this, estimation of the near-surface vertical stress component often doesn’t conform to the relationship presented in Eq. (2-2). Similar issues are also encountered in mining environments, where the estimation of vertical stress is difficult proximal to large open pit operations. This is further extrapolated to the estimation of the gravitational stress component at depth in an underground mining operation when the point of interest underlies significant volumes or pre-existing underground workings.

![Figure 2-8 – Model of topographic influence on stress trajectories (Amadei and Stephansson, 1997).](image)

As a first pass analysis, the magnitude of horizontal stress can be estimated based on the vertical stress. Amadei & Stephansson (1997) proved using an array of in situ measurements, that this estimate
should be regarded as a lower bound as it does not consider the current tectonic framework, the presence of glaciation and erosion or any relic and residual stresses. In the simplest case, it can be considered that the total horizontal stress is a result of the strain induced by the vertical loading, which is dependent on the material’s Poisson’s ratio ($\nu$). This can be shown as:

$$\frac{\sigma_h}{\sigma_v} = \frac{\nu}{1 - \nu}$$  \hspace{1cm} (2-3)

Sheorey (1994) was able to further develop this initial model to consider the depth dependency of the crust’s elastic constant, density and near surface geothermal gradient. At relatively shallow levels (< 3 km), using the average properties of the top portion of the crust, the ratio of stress was estimated as:

$$\frac{\sigma_h}{\sigma_v} = 0.25 + 7E(0.001 + \frac{1}{Z})$$  \hspace{1cm} (2-4)

Where: $E$= Young’s modulus (GPa)

$Z$= depth (m)

Figure 2-9 demonstrates how the initial stiffness approach compares with the Sheorey model as a function of depth. For illustrative purposes, it also contains empirical estimations for the upper- and lower-bound stress magnitudes based on work by Hoek & Brown (1980). It can be seen that at depths below greater than 500 m, the isotropic solution presents a reasonable estimation of global minimum horizontal stress magnitudes, however the Sheorey approach more closely predicts how the stress ratio may change with depth. It can also be postulated that any measurements of stress which exceed the prediction by the Sheorey model (shown in red) is as a result of the presence of tectonic, residual or terrestrial stresses.
2.3.2.2 Tectonic Stresses

The presence of geological structures such as folding and faulting in a rockmass is the first, and most obvious evidence that tectonic activity has, in the past, or still is currently influencing a region. It has been shown by Zoback (1992) that the largest overprinting influence on horizontal intra-plate stress is the geometry and relative sense of motion at plate boundaries. These characteristics act to either drive or resist plate motion, which causes slab pull, ridge push, trench suction and basal drag, to name a few. These forces have been summarized in Figure 2-10 and include the crustal reaction on the local scale from tectonics and the more regional tectonic stresses which act on an entire plate.
Figure 2-10 – Summary of the sources of tectonic stress (after Zoback et al. 1989)

The strongest evidence of the effect that the earth’s current tectonic framework has on horizontal stresses can be seen when examining the relative plate movement as compared to the measure of stress orientation. This is well demonstrated in Figure 2-11 where, at all locations near the mid-Atlantic spreading margin in Northern Europe, the magnitude of Maximum Principal horizontal stress is orthogonal to the margin. On a global scale, the world stress database demonstrates this same relationship.

Figure 2-11 – Tectonic stress generating mechanism at the mid-Atlantic spreading center, and the regional effect on stress orientation in Europe (after Fejerskov & Lindholm 2000).
2.3.2.3 Residual and Terrestrial Stresses

Both residual and terrestrial stresses have an influence on the state of primitive stress in the earth’s crust, although they are both deemed to be minor with respect to the overall stress state (Stephansson and Zang, 2012). This is particularly the case at depth where stress is mostly from overburden pressure and active tectonic forces. From work done at the Underground Research Laboratory (URL) in Manitoba, Cuisiat & Haimson (1992) concluded that for practical stress estimation, these features could be deemed irrelevant.

In the case of residual stress, it is termed the “self-equilibrating” stress which remains in a structure if external forces and moments are removed (Voight, 1966). In the field of rock mechanics these are termed the locked-in stresses and are at the scale of individual mineral grains. These stresses often develop during formation, re-crystallization or cementation in a rock and are generally reflective of a change in condition in the rock such as a change in stress state, eg. burial/erosion or change in temperature during metamorphism (Hyett, Dyke and Hudson, 1986).

Terrestrial stresses are caused by seasonal fluctuations in temperature, tidal changes, diurnal variations and Coriolis forces. Although deemed to be insignificant at depth, it has been shown that near surface stress estimation could benefit from the quantification of these stresses (Berest, Blum and Durup, 1992).

2.3.2.4 Glaciation Induced Stresses

Over the course of Earth’s history, much of the continental landscape has seen some form of glaciation. In the Canadian context, this has occurred as recently as 14,000-24,000 years ago during the Wisconsinan glacial event, where the Laurentide ice sheet covered much of Canada at a maximum thickness of 3 km (Dyke et al., 2002). During formation of the glacier, the main stress influences include an increased vertical stress from the weight of the mass of ice and the subsequent Poisson’s effect which this has on the horizontal stress magnitude. This causes the crust to respond by shortening through depression of the lithosphere, with some crustal flexure occurring between the point of maximum ice thickness and the glacier’s front. These features and their relative effect on the stress tensor are shown in Figure 2-12.
During the retreat of a glacier, the release of overlying load leads to crustal re-equilibration, known as isostatic rebound (Dyke et al., 2002). This is partially caused by the melting of the glacier but also by the erosion caused during glaciation. On a geological scale of time, this causes a rapid release of vertical stress, while much of the horizontal stress remains stored.

2.3.3 Limits of Stress from Frictional Faulting
An important concept when estimating stress, is the understanding that the absolute maximum magnitude of stress is dictated by the strength of the rockmass. This operates under the assumption that the crust is in a pseudo-equilibrium, meaning that existing faults aren’t displacing and no new faults are forming (Amadei and Stephansson, 1997). If this wasn’t the case, failure and a measurable release of seismic energy would occur, thus denoting an excess of crustal stress. Given the frequency of planar structures such as faults in the earth’s crust, the bounds of maximum potential stress magnitude are limited by the strength of these features above the brittle-ductile transition zone at ~3 km depth (Zoback et al., 2003a). This has been shown from the work done in the KTB borehole in Germany, where the magnitudes of stress are restricted by the frictional strength along pre-existing faults (Brady and Brown, 1985).

Assuming the Mohr-Coulomb criterion to define the strength of the rockmass, it is described by both the material’s friction angle ($\phi$) and cohesive strength ($c$). There are however two important
assumptions which must be understood when using this criterion; 1) the strength of the crust is independent of the intermediate stress component and 2) faulting occurs at a 45° angle to the direction of maximum principal stress. Jaeger & Cook (1976), then showed that the relationship between Major and Minor Principal Stress is as follows:

\[ \sigma_1 - \sigma_3 = UCS + \sigma_3 \left[ \tan^2 \left( \frac{\pi}{4} + \frac{\phi}{2} \right) - 1 \right] \]  \hspace{1cm} (2-5)

where \( UCS \) is the unconfined compressive strength of the rock as defined by:

\[ UCS = 2c \cdot \tan \left( \frac{\pi}{4} + \frac{\phi}{2} \right) \]  \hspace{1cm} (2-6)

If it is then assumed that one of the major principal stress components is in the vertical axis \( (\sigma_i=\sigma_v) \), Eq.(2-5) can be rearranged to reflect the three possible states, as defined by Anderson’s fault classification system (Section 2.3.1). In a normal faulting regime, which is characterized by \( \sigma_3=\sigma_v \), this produces an estimate of the minimum horizontal stress ratio \( K_{min}=\sigma_h/\sigma_v \) of:

\[ K_{min} = \cot^2 \left( \frac{\pi}{4} + \frac{\phi}{2} \right) - \frac{UCS}{\sigma_v} \cdot \cot^2 \left( \frac{\pi}{4} + \frac{\phi}{2} \right) \]  \hspace{1cm} (2-7)

In the case of strike-slip faulting, the maximum potential stress is defined by two stress ratios which include; \( K_H=\sigma_H/\sigma_v \) and \( K_h=\sigma_h/\sigma_v \). This takes the following form:

\[ K_H = \frac{UCS}{\sigma_v} + K_h \cdot \tan^2 \left( \frac{\pi}{4} + \frac{\phi}{2} \right) \]  \hspace{1cm} (2-8)

When the tectonic state is deemed to be in compression, the maximum state of stress is represented by the stress ratio \( K_{Max}=\sigma_H/\sigma_v \). This produces the following equation:

\[ K_{Max} = \frac{UCS}{\sigma_v} + \tan^2 \left( \frac{\pi}{4} + \frac{\phi}{2} \right) \]  \hspace{1cm} (2-9)

It should be noted that at depths greater than 200 m, the cohesive strength of a fault may be neglected, as it’s magnitude is relatively insignificant with relation to the actual amount of stress which is acting on it (Jaeger and Cook, 1979). If this assumption holds that the strength of a fault is only defined by a frictional component, the forms above can be simplified into the following:
\[
\frac{\sigma_1}{\sigma_3} \leq \left( (\mu^2 + 1)^{1/2} + \mu \right)^2
\]

(2-10)

where \( \mu \) is the coefficient of friction and the orientation of the principal stress magnitudes is dependent on the fault regime which is present at the location of stress measurement.

From lab testing on a variety of rocks Byerlee (1978) has shown that a coefficient of friction between 0.6 and 1.0 is applicable to brittle, crustal conditions. If a value of 0.6 is used in Eq.(2-10), and applied to each faulting regime, Figure 2-13 can be used to illustrate the allowable magnitudes of horizontal principal stress, provided the vertical stress is known at that location (estimated using overburden pressure). From this plot, it should be noted that if stress lies outside of each quadrant, then faulting is actively occurring as the stress state has exceeded the assumed material strength. This is shown by the inset Mohr circle diagrams and their associated points along the plot boundary.

Figure 2-13 – Stress polygons defining the potential magnitudes of horizontal stress in various Andersonian faulting regimes, for a given coefficient of friction of 0.6 and vertical stress equal to that at 3 km depth in rock with an average density of 2.3 g/cm³ (after Zoback et al. 2003b).

From the inspection of Figure 2-13, a number of intuitive relationships can be seen. The first is the presence of the 1:1 ratio line where the stress state is required to fall above this in the polygons. This represents the area where \( \sigma_1 \geq \sigma_3 \), which must always be satisfied. The point at the center of the plot labelled \( \sigma_v \) represents the isotropic stress state where \( \sigma_1 = \sigma_2 = \sigma_3 \), which must always fall on the previously
mentioned line. Above this, the polygons’ size and relative stress magnitudes are defined by the underlying assumption in Eq.(2-10), and the estimated vertical stress magnitude. From this, with some previous knowledge of the tectonic environment, the stress state can be narrowed down to the area that it must fall in. This result can then be further refined and validated through in situ measurements. The labelled points along the outer perimeter are associated with examples of stress magnitudes where the crustal stress is said to be exactly in failure equilibrium with the critical faults strength. This is shown in the inset of the figure, where each case is shown by Mohr-Coulomb circles which are exactly intersecting the failure envelope, that has a slope equal to that of the assumed coefficient of friction.

2.4 Stress in the Canadian Shield

Within the Canadian Shield, many of the attempts at stress estimation have come from various hardrock mining operations. As a result of this, a relatively wide variety of measurements have been taken at different depths to produce an average gradient of stress in this geological region. Most mineralization within the Canadian Shield is found in the east-trending volcanic-sedimentary orogenic belts, which is largely comprised of Archean igneous rocks that have been repeatedly intruded by felsic and mafic volcanics associated with orogenic activity (Arjang and Herget, 1997). This same deformation is what also brought much of the large scale mineralization concentrated along major regional faults and shear zones. One notable temporal exception to this is the occurrence of the Sudbury impact structure which was the driver for mineralization throughout that region (as discussed in Section 2.1). Upon inspection of the volcanic complexes, the closely spaced folding indicates that the predominant deformation was through compressive folding, which has resulted in a shortening in the North-South direction with later stage compression in the East-West axis (Arjang and Herget, 1997).

The basis for the estimation of virgin stress throughout the Canadian Shield has come from overcoring strain cell measurements from 39 underground mines. This was completed by Herget (1988), where 214 ground stress tensor measurements, from 12-2134 m depth, were statistically reviewed to determine magnitude and orientation of principal stresses. Although some scatter of stress orientation
exists between subsequent measurements, as can be seen in Figure 2-14, the average orientation of the Maximum Principal Stress in the Canadian Shield is approximately East-North-East. This result is validated by the indicated East-West compression shown in the folding of the volcanic complexes and from work done by Zoback et al. (1989) and Zoback (1992) on a broader scale across most of the mid-plate region of North America.

Figure 2-14 – Location of overcoring measurement locations with direction of Maximum principal stress throughout the Canadian Shield (Herget, 1987).

From the magnitudes of stress, Arjang & Herget (1997) defined the stress-depth relationship based on ratios of vertical stress to both the horizontal stress magnitudes, taking the following forms:

\[
\sigma_v = 0.026 \cdot \text{depth} \quad (\text{Overburden Pressure}) \quad (2-11)
\]

\[
\frac{\sigma_H}{\sigma_v} = 7.44 \cdot \text{depth}^{-0.198} \quad (2-12)
\]

\[
\frac{\sigma_H}{\sigma_v} = 2.81 \cdot \text{depth}^{-0.120} \quad (2-13)
\]
From this relationship it can be seen that both stress ratios decrease with depth and, at any location exceeding approximately 1000 m, the ratios approach 2.0 for $\sigma_H/\sigma_v$ and 1.0 for $\sigma_h/\sigma_v$. Diederichs (1999) however showed that these fits lacked in considering the assumption that all stress ratios must converge to unity at great depths in a mid-continental compressive setting. This is caused by a reduction in shear resistance due to the increase in temperature/pressure and inherent plasticity. As a result, the fits were modified to consider this, thus taking the new forms shown in Eqs.(2-14) and (2-15). A plot of these fits with the original data, as compared to the global trends created by Hoek & Brown (1980) and the Sheorey Model can be seen in Figure 2-15.

\[
\frac{\sigma_H}{\sigma_v} = 1 + \frac{25}{\sqrt{depth}} \tag{2-14}
\]

\[
\frac{\sigma_h}{\sigma_v} = 1 + \frac{8}{\sqrt{depth}} \tag{2-15}
\]

An important relationship to note is the fact that at almost all points with depth, the ratio of Major Principal stress to vertical stress plots above the Sheorey Model and therefor follows the isotropic model presented above in Section 2.3.2.1. This is further proof of the effect which tectonic and glacial loading have on the overall estimation of stress. In Figure 2-15, it can also be seen that the large majority of minor horizontal stress to vertical stress ratios are above one, thus denoting a compressional reverse/thrust faulting regime according to Anderson’s faulting theory. In the cases where the ratio is below one, this denotes a situation where vertical stress is the intermediate principal stress component, thus indicating a strike-slip dominated environment.
Figure 2-15 – In situ stress relationships for the Canadian Shield with a) overcoring measurement points and associated stress ratios (Arjang and Herget, 1997) and b) estimation of stress magnitudes with depth using Eqs. (2-14) and (2-15), (Diederichs, 1999).
2.4.1 Local Stress Influences

Although currently the Sudbury Igneous Complex is found in a passive mid-continental setting, the driving influence on the stress state is from tectonic activity in both the Pacific and Atlantic Oceans (Zoback et al., 1989; Zoback, 1992; Reiter et al., 2014). Along the Western coast of Canada is the Cascadian subduction zone where the Juan-de-Fuca Plate is sliding under the North American Plate. Approximately 1500 km outboard of the East coast, is the mid-Atlantic Spreading Center which is actively causing the divergence of North America from the European and African continents (Reiter et al., 2014). This tectonic activity is currently driving the roughly East-West stress orientation throughout much of the passive mid-continent, although as previously mentioned, the local stress tensor surrounding the Sudbury Basin is quite variable as a result of the complex deformational history and presence of related structures.

Perhaps the largest overprinting stress influencer proximal to the Sudbury Basin is from the meteor impact that created it. Post-impact, it is thought to have caused a large degree of crustal thinning and mantle upwelling with reactivation of regional scale basement faults. Given the ‘bowl-like’ geometry of the basin (Figure 2-2), at any point around its perimeter, the stress tensor is defined by local structural features. This has been consistently demonstrated from structural fabric in the country rock surrounding the basin (Cowan, Riller and Schwerdtner, 1999; Santimano and Riller, 2012) and through in situ stress measurements at depth in mining settings (Snelling et al. 2013; Trifu & Suorineni 2009; Walton et al. 2015). Proximal to the Victoria project, the most prominent second-order stress influencer is the Creighton Fault, which is thought to have originated as a normal fault, with subsequent repeated reverse-sense reactivation during the Penokean Orogen and dextral-oblique strike-slip displacement during the Neoproterozoic (Mungall et al., 2004; Snelling, Godin and McKinnon, 2013). From previous measurements of displacement and associated structures near the fault shown by Cowan et al. (1999), the minor principal stress ($\sigma_3$) is horizontal and parallel to the fault’s strike. More distal to this in the Southern Range and throughout much of the Canadian Shield, $\sigma_3$ acts in the vertical direction (Herget, 1987, 1988; Arjang and Herget, 1997).
2.5 Borehole Scale Stress Determination

As previously mentioned, the true magnitude of in situ stress can’t be measured, rather it can be inferred based on the measurement of strain, knowing the physical properties of the rockmass. In the case of the determination of borehole scale stress, this is done by disturbing the stress field surrounding the opening by a predictable amount (Kirsch Equation), which then can be used to estimate what the original virgin stress state may have been. This relationship is shown in Eq.(2-1), where stress at any point in the crust is a function of the in situ stress and the disturbance to that stress field which must be created to impart some form of measurable strain.

Amadei and Stephansson (1997) showed in their work that the prediction of the disturbed stress around a borehole can be done with an associated error of 10-25% provided the rockmass is deemed to be linearly elastic and homogeneous. Given this, it led them to the conclusion that in “structurally complex rocks” the estimation of in situ stress from borehole investigations should not be done. As a result of the volatility associated with most ore body emplacement styles, the occurrence of perfectly crystalline and isotropic rock throughout a mining project often isn’t seen. This leads to some associated uncertainty with any measurement at the scale of a borehole. Despite this, at early stages in mining or civil infrastructure planning, almost all geotechnical site investigation data must be collected from surface drilled boreholes.

To date, the most commonly used methods of stress measurement include either borehole overcoring or hydraulic fracturing. Another form of measurement is more indirect than the previous two and involves characterizing the cross-sectional elongation of the borehole from preferential rock failure during drilling (Haimson and Song, 1998). At the very least the occurrence of breakout denotes overstressed conditions, but can also be used to back-out the original stress state. Other indirect measurements of stress such as using core disking have also been proposed, although many authors have questioned its practical validity (Holcomb, 1993; Amadei and Stephansson, 1997; Lavrov, 2003). From the geological and measurement uncertainty associated with any borehole-scale investigation, it is
frequently recommended that for proper stress state characterization, a combination of methodologies should be used to validate each measurement (Hudson, Cornet and Christiansson, 2003).

2.5.1 Hydraulic Borehole Testing

The use of hydraulic fracturing (hydrofrac) was first conceived in the oil and gas industry where it is used to create new fractures in the reservoir to artificially increase the formation’s permeability (Haimson and Cornet, 2003). In the context of stress estimation, hydrofrac methods can either be done by inducing a new fracture in the rockmass or by hydraulically testing pre-existing fractures (HTPF) and is a relatively efficient way of estimating the biaxial stress condition. This firstly involves isolating an interval of the borehole, then pressurizing the contained area with fluid until a fracture is generated or an existing one is dilated and/or sheared.

2.5.1.1 Hydraulic Fracturing

The beginning of test set up for a hydrofrac test is to isolate the interval of interest using a straddle packer (Figure 2-16a). The size of the zone should be no less than 6 times the hole diameter, to allow for a realistic propagation of the induced discontinuity (Haimson and Cornet, 2003). Once the target zone is established, multiple cycles of pressurizing/depressurizing through fluid injection occurs where the necessary parameters for stress estimation are collected. The orientation of the fracture must then be logged following the test by either using an impression packer or geophysical tool such as a borehole camera or televiewer (Haimson and Cornet, 2003). This is important as the orientation of the fracture relative to the borehole is indicative of the orientation of the assumed Minor Principal Stress.

During fluid injection, the first cycle is known as the breakdown cycle where the peak applied pressure ($P_w$) is achieved and is denoted in Figure 2-16b as the breakdown pressure ($P_b$). In theory, it is this point that corresponds to the formation of the hydraulic fracture. As soon as this occurs, the crack propagates quasi instantaneously as it connects to the in situ permeability. When this happens, the constant flow rate from the test is not enough to sustain a constant pressure due to the increasing crack size, causing the pressure to drop rapidly. Once injection is terminated (shut-in), the pressure decreases
until it reaches a constant shut-in pressure ($P_{si}$), which denotes the pressure needed to jack open the artificial fracture. Pressure is then bled off from the system, thus closing the crack and isolating it from the surrounding rock’s permeability. A second and third injection cycle is often performed, where the maximum re-opening pressure ($P_{r}$) is theoretically less than the breakdown pressure as the fracture has already been formed. This cycle is continued until shut in is once again reached and measured to compare with the first $P_{si}$ measurement.

![Diagram showing downhole test setup and injection sequence with key thresholds](image)

**Figure 2-16** – The technique of hydraulic fracturing for stress estimation showing a) downhole test setup and, b) injection sequence with key thresholds (after Rutqvist et al. 2000).

### 2.5.1.1.1 Measurement Principle and Assumptions

When the minimum principal stress is assumed to be in the horizontal axis (normal or strike-slip faulting regime), this testing induces a vertically oriented fracture trending in the direction of $\sigma_{H}$ and perpendicular to the $\sigma_{h}$ axis in a vertical borehole (Figure 2-17). In this fracture orientation where the evaluation of stress magnitude is well established (Lee and Haimson, 1989; Haimson and Cornet, 2003). Using a pressure-time graph such as the one shown in Figure 2-16b, the following information can be deduced:
\[ \sigma_h = P_{si} \]  

(2-16)

where the minimum horizontal stress occurs normal to the vertical crack with its magnitude being equal to the shut-in pressure, which is the pressure required to maintain the opening of the fracture.

\[ \sigma_H = T + 3\sigma_h - P_b \]  

(2-17)

where the maximum horizontal stress occurs along the strike of the vertical crack. Its magnitude is deduced from the Kirsch solution of stress around a circular opening and the tensile strength from laboratory testing, \( T \). The assumption in Eq. (2-17) is also that interaction between testing fluid and pore fluid is insignificant. In a saturated rock, with an appreciable permeability, the influence of pore pressure, \( P_o \), can be accounted for by:

\[ \sigma_H - P_o = T + 3(\sigma_h - P_o) - (P_b - P_o) \]  

(2-18)

In the case where core can’t be collected, or lab testing is deemed to be unreliable, the relationship between re-opening pressure, \( P_r \) can be substituted for the break-down pressure, \( P_b \). Given some associated concerns with reliability in lab scale tensile strength tests, this approach is often used in practice (Ito et al., 1999). The use of re-opening pressure takes the following form:

\[ T = P_b - P_r \]  

(2-19)

\[ \sigma_H - P_o = 3(\sigma_h - P_o) - (P_r - P_o) \]  

(2-20)
Figure 2-17 – Stress estimation from hydrofrac test results showing, a) determination of minor principal stress, and determination of major principal stress if b) reliable lab testing of tensile strength is available or c) the re-opening pressure if strength is not known.

As outlined in the ISRM suggested method for rock stress estimation (Haimson & Cornet 2003), a number of considerations doing a hydrofrac test include:

- There is no limit in depth of where a test can be done, provided the borehole remains un-blocked and the rock remains in an elastic and brittle state.
- The conventional interpretation of hydrofrac tests can be done when one of the principal stress axes occurs parallel to the axis of the borehole and is contained in the induced fracture’s orientation.
- The prediction of principal stress magnitudes occurs in the cross-sectional plane of the borehole and assumes that the rockmass is linearly elastic, homogeneous and isotropic. It can also consider the presence of pore pressure (of particular interest in oil and gas testing) and requires knowledge of the tensile strength of the rock.

When completing a hydrofrac test, a number of limitations exist which must be taken into consideration. Many of these relate to the validity of the underlying assumptions which are described above. In brief, these include:
- Poroelastic effects: During the test, it is assumed that the injected fluid does not penetrate the surrounding rockmass. Although this has been shown to hold true in crystalline rocks, in a porous medium such as sandstone, infiltration of fluid has been shown to contribute to the stress field surrounding the borehole (Ito et al., 1999).

- Inducing fractures in a reverse faulting regime: This produces a sub-horizontal radial fracture in theory, which is only indicative of the vertical stress component. As this is often assumed by the weight of overburden the test result yields limited information of the stress state. In deep wells where the vertical stress is the minor principal stress, it has been shown that axial fractures can still form, although they are not indicative of the shut-in pressure, \( \Psi \), as it is thought that away from the borehole wall they tend to preferentially rotate back to a horizontal nature (Evans and Engelder, 1989). This then nullifies one of the key underlying assumptions doing a hydrofrac, that the induced crack is perfectly planar.

- Presence of pre-existing discontinuities: This could be natural fractures which cross the borehole or ones which don’t, but intersect the induced crack at some distance from the borehole wall. The former of the two is less of an issue as it can be accounted for during downhole surveys, however the ones which don’t intersect the borehole can only be inferred, if at all, from anomalous test results. Such features are not limited to natural fractures but also could include other incipient planes of weakness such as sedimentary bedding or pervasive foliation (Ljunggren et al., 2003).

- Tensile strength exceeds fluid capacity: If the tensile strength of the rock at the measurement location exceeds that of fluid pressure in the hole. In this case, no fracture can be generated for the evaluation of stress. This has been noted to be particularly problematic in crystalline rock of uniform grain size, such as at AECL’s Underground Research Laboratory (URL) where borehole-parallel fractures were difficult to achieve through hydrofrac tests. This led to the usage of the testing of pre-existing fractures for stress delineation, as described below in Section 2.5.1.2-(Thompson and Chandler, 2004).
2.5.1.2 Hydraulic Testing of Pre-Existing Fractures

As a variation to the conventional method of hydraulic fracturing, the testing of existing discontinuities can also yield meaningful stress results. The feature of interest must firstly be identified based on core logging results or through the use of a borehole televiewer. The test procedure is similar to that of a hydrofrac test, where the interval of interest is isolated using packers prior to fluid injection. Throughout the first test and subsequent cycles, the objective is to determine the borehole pressure which exactly matches the normal stress on the fracture. Once this stress is exceeded, the joint opens and is denoted in test results by a significantly higher flow rate required to keep dilation on the fracture. It is this shut-in pressure, $P_{si}$, which is equal to the normal stress acting on the joint.

As jointing commonly acts at an angle to the principal stress axis, the magnitude of stress that is clamping the joint is not indicative of a principal stress magnitude, rather it is one of the components of stress, $\sigma_\theta$, with a given orientation defined by the normal vector to the joint plane. In comparison to the hydrofrac method where, in theory, the normal stress on the induced crack is equal to the minimum horizontal principal stress, in an HTPF test the normal stress is a function of all three principal stress magnitudes (Figure 2-18). As the HTPF method is considered a quasi-static test, it operates under the assumption that the opening of the fracture occurs in a way which does not cause the rotation of the fracture plane preferentially towards the principal stress axis (Ask et al., 2009).
Given this 1-dimensional estimation of stress, a minimum of six tests on variably oriented joints is required to resolve the entire stress matrix (as shown in Section 2.1). Haimson and Cornet (2003) argued that a minimum of eight tests should be conducted to account for the uncertainty associated with estimating the joint orientation and normal stress magnitude. If, however, the assumption that the vertical stress is equal to the overburden pressure, this reduces the amount of unknowns which need to be solved for. In this case, if each subsequent test is over a relatively dispersed depth interval (>50 m), then the change in stress as a result of this must be accounted for during the processing (Haimson and Cornet, 2003).

To fully delineate the stress field from each individual test, a system of linear equations is made relating the orientation and magnitude of normal stress from each test. In the case where multiple fractures exist in a tested interval, a downhole survey should be done to determine which fracture opened as a result of the fluid pressure. The system of equations is then solved for using the Misfit Function, which defines the difference between observed and computed values of stress. The chosen solution is the

\[
\sigma_n = f(\sigma_h, \sigma_l, \sigma_v)
\]
stress model which acts to minimize the misfit function—i.e., the model which most closely represents all test values. The function takes the following generalized form:

\[
\phi_j = \sum \frac{|\sigma_{ni}^{mi} - \sigma_{ni}^{ci}|}{|\delta_{\sigma_{ni}} - \delta_{\sigma_{ni}^i}|} \tag{2-21}
\]

Where: \(\phi_j\) = misfit associated with the model \(j\)

\(\sigma_{ni}^{mi}\) = measured normal stress for the \(i^{th}\) test

\(\sigma_{ni}^{ci}\) = computed normal stress from model \(j\)

\(\delta_{\sigma_{ni}}\) = standard deviation associated with the normal stress measurement from the \(i^{th}\) test

\(\delta_{\sigma_{ni}}^i\) = change in normal stress from a small perturbation of the fracture plane orientation

As can be seen from Eq. (2-21), the determination of fit is based on uncertainties from the normal stress measurements and fracture plane orientations. For the perturbation in discontinuity orientation, both the dip and strike are systematically rotated about their axes to evaluate how the error in measurement of these functions influences the final estimation of the complete stress tensor. This is done using a generic algorithm such as a Monte Carlo simulation, where in one iteration the best fit is selected and then compared with a new set of orientation inputs which are statistically determined. This simulation continues until a certain confidence level is achieved in the misfit result, which is indicative of the most probable stress state (Cornet et al., 1997).

The use of HTPF is most appropriate over the previously discussed hydrofrac method in a number of settings:

- When the rockmass has a joint frequency such that an intact portion of the borehole can’t be isolated by packers for a conventional hydrofrac test,
- When the intact rock is of a strength that can’t be exceeded by fluid injection pressure, thus unable to form a new fracture for evaluation of stress,
• When the area is in a reverse faulting regime ($\sigma_v = \sigma_3$), a hydrofrac creates a borehole perpendicular fracture which can only resolve the magnitude of vertical stress without giving an indication of the horizontal stress state, and

• If the borehole is at an appreciable angle to the principal stress field, thus inducing en-echelon cracks relative to the borehole axis.

### 2.5.2 Overcoring Methods

The principle behind all overcoring methods is based on observing the response of a sample of rock when it is removed from the stress field. As a simplification, this is done by measuring the accumulated strain to the sample when the stress state is zero. The process of overcoring in a borehole is shown in Figure 2-19, which involves drilling a small-diameter hole where the strain measuring tool is placed and adhered to the wall. A larger diameter hole is then progressed over the location of the strain device, which relieves the stress on the sample of rock surrounding the gauge, allowing for the simultaneous measurement of strain accumulation during the drilling procedure. Once overcoring has been completed, the overcored sample is recovered and tested in a biaxial cell to determine the pertinent elastic properties of the rock for use in solving Eqs.(2-22) to (2-24) below. During testing, the sample is loaded in a step-wise manner and then subsequently unloaded in the same fashion, with the strains being measured throughout the test (Amadei and Stephansson, 1997).
2.5.2.1 Overcoring Assumptions

For the theory of overcoring to hold, a number of key assumptions regarding rockmass conditions need to be made. The most important of these include that the testing medium is homogeneous, isotropic and linearly elastic—like many of the other stress estimation methodologies. It also needs to be accepted that the probe is mounted far enough from the bottom of the borehole such that no significant stress or strain gradients exist across the axis of the test device (Amadei and Stephansson, 1997). This allows for the measured deformation of the overcore during stress relief to be the same in magnitude from what is created by the in situ stress field, but of opposite sense (Hirashima and Koga, 1977). If it is thought that
these assumptions have been adequately satisfied, then the following equations can be use to resolve the stress tensor:

\[
    u_r = \frac{1 + \nu}{2E} r \left( 1 + 4(1 - \nu) \frac{R^2}{r^2} - \frac{R^4}{r^4} \right) \left\{ \frac{1 - \nu}{1 + \nu} \left[ (\sigma_x + \sigma_y) \cos 2\theta + 2\tau_{xy} \sin 2\theta \right] + \frac{1}{2} \frac{\nabla \cdot \sigma}{\nabla \cdot \sigma} \right\} + \\frac{\nabla \cdot \sigma}{\nabla \cdot \sigma} \\
    u_\theta = -\frac{1 + \nu}{2E} r \left( 1 + 2(1 - 2\nu) \frac{R^2}{r^2} + \frac{R^4}{r^4} \right) \left\{ (\sigma_x - \sigma_y) \sin 2\theta - 2\tau_{xy} \cos 2\theta \right\} \\
    u_z = \frac{1 + \nu}{2} 2r \left( 1 + \frac{R^2}{r^2} \right) \left\{ \frac{\tau_{yz} \sin \theta - \tau_{zx} \cos \theta}{\tau_{yz} \sin \theta - \tau_{zx} \cos \theta} \right\} + \frac{\nabla \cdot \sigma}{\nabla \cdot \sigma} \frac{z}{1 + \nu} \left\{ \sigma_z - \nu(\sigma_x + \sigma_y) \right\}
\]

Where:

\( R \) = borehole radius
\( r \) = distance to the measurement point
\( E \) = Young’s Modulus
\( \nu \) = Poisson’s ratio
\( \sigma_i \) = far-field stress component \( i \)
\( \theta \) = location of strain gauge in the borehole

2.5.2.2 Measuring Instrument Types

A number of overcoring gauges exist which produce varying levels of data quality in different testing conditions. Broadly speaking, there are three main categories of cells which include the United States Bureau of Mines (USBM) deformation gauge (Hooker and Bickel, 1974), the Doorstopper gauge (Leeman, 1969) and what are called soft inclusion cells. Both the Doorstopper and USBM gauges are only able to resolve the stress field in the cross sectional plane of the borehole as a result of the orientation of the strain gauges in the measuring device. Although somewhat of a limitation, when applied in a vertical borehole, the axial component of stress can be approximated by the overburden pressure. In
the case of the soft inclusion cells such as the CSIRO gauge, up to 12 strain gauges are used in the measuring device and are oriented in the axial and tangential sense, and at 45°. This allows for the resolution of the complete stress tensor. As previously mentioned, each of these gauge types require a dry hole to allow for the adhesive to bond properly between the device and the borehole wall (excluding USBM) and can only operate up to approximately 50 m downhole (Ljunggren et al., 2003).

To overcome these limitations, the Atomic Energy of Canada Ltd. (AECL) developed the deep doorstopper gauge (DDGS) with the Swedish State Power Board (SSPB) creating their own variation, the Borre probe. In both cases, the systems were intended for deep, highly stressed ground and consisted of a rosette of strain gauges which attach to the bottom of a flat borehole. The adhesive used for both instruments is also compatible with water, with each having a maximum operational depth of 1000 m. Despite this, the maximum successful test to date has been 620 m (Ljunggren et al., 2003). In the case of all overcoring devices, the associated error occurs as a result of the assumptions regarding the rock’s rheological behaviour with strong limitations occurring in anisotropic rock.

An example of raw data from an overcoring test can be seen in Figure 2-20, using a Borre Probe at the Äspö Hard Rock Laboratory (HRL) in Sweden. This reflects the typical test process including firstly installing the gauge, flushing the hole with water and completing the actual overcoring. It can also be seen that temperature is constantly logged throughout the test to correct the observed strains for the associated effects.
2.5.3 Core Disking

During the diamond drilling process, the removal of core from the in situ stress field can induce tensile fractures given the proper stress environment. This produces repeatable “breaks” in the core, resulting in the formation of disks with a thickness that has been shown to be related to the rock’s tensile strength and the stress state where it occurs (Jaeger and Cook, 1963; Obert and Stephenson, 1965; Durelli, Obert and Parks, 1968; Stacey, 1982). Despite the understanding of the mechanisms which lead to the occurrence of core disking, many still view this as a purely qualitative way of assessing stress (Doe et al., 2006). As part of the work done at the URL, various boreholes extending away from the research drift were completed to assess how disking thickness changes as a function of radial distance from the opening (Lim and Martin, 2010). As can be seen in Figure 2-21, the change in stress magnitude away from the excavation greatly
affects the intensity of disking, where over the span of 2 m, the disk size changed from being millimeter scale fragments to almost fully intact by the end of the hole.

![Bore hole core disking image]

**Figure 2-21** – Core disking observed in a borehole extending radially from an excavation on the 420 level of the URL. From the top to end of the hole, it can be seen how stress concentration as a result of the excavation influences disk size, decreasing away from the opening (Lim and Martin, 2010).

2.5.3.1 Mechanism of Core Disking

This work by Lim and Martin (2010) also modelled the evolution of stress at a reference point along the axis of a borehole assuming rock and stress conditions present at the research site. The conceptual model setup is shown in Figure 2-22, with the results of the evolution of stress path from the three points of interest, along both principal stress axes and through the core axis. The three dimensional elastic analysis shows that at the beginning of the drilling process, the plane of interest is fully in compression. Gradually, as the bit approaches the reference location, the initial compressive stresses become tensile at $B_2$ and $C_2$, although this stress concentration is only localized as the point $A_2$ remains compressive. This indicates that the occurrence of disking initiates along the core axis once the tensile envelope is exceeded and then gradually propagates outward to points $A$ and $B$.

From this work and by a number of other authors, it was concluded that, based on the observed failure mechanism, the rock’s tensile strength was not the sole controlling metric driving core disking.
The thickness of the disk also played a role as it acts to provide some form of confinement in the axial sense of the core (Haimson and Lee, 1995; Kaga, Matsuki and Sakaguchi, 2003; Lim and Martin, 2010).

Figure 2-22 – Stress path taken along three monitoring lines (shown in top left model illustration) during elastic modelling of core disking formation in Lac du Bonnet Granite. The Hoek-Brown failure envelope-(Martin, 1997) and crack initiation ($\sigma_{ci}$)-(after Lau & Chandler 2004) from lab testing of the rock type are shown with the model’s far field stress being equal to that at the URL (Lim and Martin, 2010).

Much of the early work on disking failed to consider how the axial length of the core would impact the occurrence of tensile failure and subsequent predictions of stress conditions in situ. More recent work has been done in settings where a high confidence of in situ stress conditions exist. This has allowed for more careful consideration using lab based testing and numerical modelling of how core length affects disking. This has produced the following relationships for the prediction of in situ stress:
\[ \sigma_1 = A + \frac{B}{e^{(\frac{t}{c})}} \]  \hspace{1cm} (2-25)

where \( t \) is the core disk thickness, and A, B, and C are curve-fit parameters as derived by Lim and Martin (2010), on work done at the URL.

It can be noted that this formula fails to consider the tensile strength of the rock, however for this study, the model inputs included the rock’s strength thus this relationship is only applicable to the granodiorite observed in the study. These authors however alluded to the fact that the formula would remain the same across different rock types and inherent strengths, however the fit-constants would change to reflect this. The following relationship developed by Kaga, Matsuki and Sakaguchi (2003) took a more generalized approach of considering both the core thickness and the tensile strength of rock

\[ \frac{\sigma_{tc}}{\sigma_m} = -A + B \frac{\sigma_x}{\sigma_m} - C \left( \frac{\sigma_x}{\sigma_m} \right)^2 - D \frac{\sigma_x - \sigma_y}{\sigma_m} \]  \hspace{1cm} (2-26)

where \( A, B, C \) and \( D \) are fit coefficients dependent on the core length, \( \sigma_{tc}/\sigma_m \) is the critical tensile stress (function of strength) normalized to the mean cross sectional stress on the core disk and \( \sigma_x, \sigma_y \) is the stress acting at the boundary of the core in the respective \( x \) and \( y \) axis.

From polyaxial testing of the Lac du Bonnet granite in a lab setting, Haimson and Lee (1995) were able to show how the thickness of observed disks was able to predict stress magnitudes around the borehole. When the relationships are applied to the observed core disking at the URL site, the three various methods for prediction of stress were compared by Lim and Martin (2010). As is evidenced in Figure 2-23, the results from each method show a remarkably large spread in predicted stress. Despite the work by Kaga et al. (2003) and Haimson and Lee (1995) being in relatively good agreement, the more recent fits of the URL data by Lim and Martin (2010) do not reflect this congruence. Despite these authors being in agreement regarding the mechanisms driving disking, this disparity in results supports the general consensus that disking should remain a purely qualitative way of interpreting the stress state.
Figure 2-23 – Plot of disk thickness normalized to core diameter (t/D) versus the maximum stress normalized to tensile rock strength (σ₁/σᵣ) from the relationships proposed by Kaga, Matsuki and Sakaguchi (2003), Haimson and Lee (1995) and Lim and Martin (2010)—denoted as “URL field data”. The URL data denotes in situ core observations of disk thickness, while the other two sets represent predicted magnitudes from either lab testing or numerical modelling by the authors (Lim and Martin, 2010).

2.5.4 Review of Borehole Deformation and Breakout

The development of down-hole geophysical tools over the past several decades, has allowed for the continuous review of the magnitude of deformation with depth along a borehole (Zoback et al., 2003a). The continuity of measurement associated with this technique provides a distinct advantage over hydraulic fracturing or overcoring, that can both only provide point estimates of stress. In the case of an absence of formal borehole breakout, the upper bound of stress can still be estimated as a function of the strength of the rock surrounding the borehole.

The first published quantification of borehole breakout was by Carr (1974), through six boreholes in the Yucca Flat, Nevada. Although the recognition of stress magnitude was not made, the author did note how the orientation of breakout was remarkably congruent with the direction of minimum horizontal
principal stress. This result initially had very little traction until Bell & Gough's (1979) work using a database of oil wells in Alberta confirmed a consistent orientation of breakout. Laboratory work was then conducted by Mastin (1984) and Haimson & Herrick (1985), who confirmed that stress-derived borehole breakout in a vertical hole, is aligned with the $\sigma_b$ direction. Since that point in time, the use of borehole breakout for the derivation of stress orientation has been well documented, with 28% of the crustal stress directions on the World Stress Map being derived from borehole investigations (Zoback, 1992).

Despite this success, there remain large amounts of uncertainty regarding the use of borehole breakout for stress estimation. This is largely related to the uncertainty of the borehole condition at the time of breakout related to heterogeneity, anisotropy and the inherent time dependent nature of rock degradation (Amadei and Stephansson, 1997). In a laboratory setting Haimson & Herrick (1985) were able to find direct correlation between the magnitude of breakout and the far-field stress magnitudes.

The condition of stress surrounding a borehole caused by redistribution of stress is highly dependent on the mechanical properties of the surrounding rock. These properties can significantly vary based on the presence of features such as jointing, veining or micro defects at the grain-size scale. It has largely been shown, in homogeneous conditions, that the progression of breakout is dependent on porosity, mineralogy, grain size and the composition of the inter-granular matrix (Haimson, 2007). As a first pass analysis, the stress in the vicinity of a borehole in a perfectly elastic and isotropic medium can be estimated using the well-known Kirsch solution. From this it can be deduced that the location of maximum tangential wall stress around an infinitely long borehole is located at $90^\circ$ to the maximum in-plane stress direction. This creates the relative geometry shown in Figure 2-24, as defined by the breakout depth and opening angle, where breakout occurs along the Minor Principal Stress axis.
2.5.4.1 Breakout Measurement Techniques

The cross-sectional area of a borehole is, in practice, determined either through the use of mechanical calipers or with a high-resolution geophysical method such as acoustic or optical televiewers (ATV/OTV). Often times both a caliper system and televiewer are used in conjunction with each other to validate the results and assist in centering the ATV/OTV within the borehole.

In the case of mechanical calipers, they either come as a 3- or 4-arm system, which are capable of measuring change in borehole shape continuously along a borehole. The setup of a 4-arm system is shown in Figure 2-25, where common stress induced (breakout) and drilling induced profiles (key seat and washout) are shown based on the deviation of each set of arms. One obvious setback of this system is that if the arms aren’t properly aligned to the orientation of maximum breakout, then this measurement technique may underestimate the true depth of breakout while also being unable to map the angular extent of breakout. Without televiewer data to validate the results, calipers can only deduce whether breakout has occurred or not, without actually quantifying the magnitude of failure.
Figure 2-25 – Common borehole geometries measured using mechanical calipers, as expressed using the 4-arm caliper system (Ask et al. 2015, after Plumb & Hickman 985).

The use of televiewers has been extensively used in industry when evaluating the orientation and extent of borehole breakout. In the case of an OTV, this takes continuous images of the borehole wall around it’s circumference. A notable drawback of this method is the fact that if the borehole is filled with opaque fluid from drilling muds or groundwater then the quality of the image is drastically reduced. Even in ideal cases, the results from an OTV survey is unable to determine the depth of breakout from images and therefore is only of value in reviewing the extent of breakout. This has led to the popularity of using an acoustic televiewer survey to fully quantify breakout. The measuring tool continuously emits an acoustic signal at a point along the borehole wall with the returning signal being recorded for both its travel time and wave amplitude. From this, a false image of the borehole wall can be created to evaluate both the depth and angle of breakout, in addition to other important geological features (Figure 2-26).
As previously discussed, it has been shown that the shape of breakout can be directly used to infer the far-field stress state. Given the continuity of data collection that the use of ATV represents, a strong understanding of rockmass characteristics as a function of depth is required to adequately determine stress magnitude. Numerical methods can then be used to back analyze the stress state for each individual breakout profile or a batch approach can be taken given a set of material property generalizations. This was the approach taken by Walton et al. (2015) for the creation of a stress profile with depth in a borehole, and is the same methodology used throughout this dissertation, as discussed in detail throughout Chapter 4. For a complete summary of previously developed analytical solutions of stress from breakout, and the mechanisms leading to borehole damage, Chapter 5 should be referred to.

2.5.5 Summary of Measurement Types

From each of the previously described methods of downhole stress determination, different levels of knowledge regarding the state of stress can be acquired. This has been summarized in Figure 2-27, in tensor form for each measurement type. This is assuming ideal test conditions for each and that the
measurement is taken at one location. Ultimately the goal of each testing campaign is to fully resolve all components of stress. An example of this is the HTPF test where only one component of stress acting normal to the dilated fracture is determined from a single test. However conducting multiple tests on discontinuities of varying orientation ultimately yields an idea of the entire stress tensor, rather than just one component.

In many cases, the missing portion of the tensor is the vertical stress, which can be assumed from the weight of overburden. An example of this is the completion of a hydrofrac or borehole breakout analysis in a vertical borehole, where the in-plane stress components are determined from testing results and the final stress, acting axially to the borehole can be calculated knowing the overlying density of the rockmass.

![Figure 2-27 – Summary of tensor stress components from each downhole measuring technique.](image-url)
2.6 Brittle Failure Mechanics

Given the observations from intact laboratory testing that will be outlined in Chapter 3, it can be concluded that the predominant mode of failure through intact rock along the length of the pilot hole is through brittle-extensional cracking (as justified therein). This deformation mechanism will therefore be the hinge of many modelling inputs throughout this body of work and will be outlined in this section.

The occurrence of brittle failure is defined by the growth and coalescence of cracks at the grain scale, in both laboratory and excavation settings. This has been extensively recognized that even in a compressive field, these extensile cracks occur as a result of internal heterogeneities in the rock (Griffith, 1921; Myer et al., 1992; Lee and Haimson, 1993; Diederichs, 2007), and was first quantified in detail as developed by the theory of rupture by Griffith (1924).

2.6.1 Brittleness in Rock

Despite the understanding of the processes leading to brittle failure, the definition itself is mostly qualitative. The opposite type of rheological behaviour of a material would be ductility, which is defined as the ability of a substance to incur large magnitudes of inelastic strain without loss of its ultimate strength. This led Hetenyi (1966) to then simply define brittleness as the lack of ductility in a material’s behaviour. Perhaps the greatest reason for this ambiguity is the fact that brittleness in rock depends not only on the material properties but also on the geometry, size, confinement and loading rate for the area of interest (Hajiabdolmajid and Kaiser, 2002). Given the relationship between these factors and a material’s intrinsic brittleness, it is easy to understand why many geomaterials demonstrate such behaviour. Bishop (1967) was the first to introduce a brittleness index ($I_B$) which considered the peak and post-peak behaviour of a material in terms of:

$$I_B = \frac{\tau_f - \tau_r}{\tau_f}$$  \hspace{1cm} (2-27)

where $\tau_f$ is the peak- and $\tau_r$ is the residual-strength.

From this first attempt, it can be seen that no consideration of the pre-peak material behaviour is made in the definition of brittleness. Hajiabdolmajid and Kaiser (2002) said that the description of
brittleness should not only represent the “severity” of strength loss following its peak magnitude, but also needs to consider how a material reacts to plastic deformations which occur prior to ultimate failure. This concept is illustrated in Figure 2-28, where a number of stress-strain relationships are presented showing different post-peak material behaviour despite having the same magnitude of strength loss. This is important as different rates of strength mobilization/decay results in varying stress redistribution.

Figure 2-28 – Demonstration of numerous stress-strain curves which demonstrate materials with the same brittleness according to Eq. (2-27). Despite this, each material demonstrates a varying rate of strength loss (Hajiabdolmajid and Kaiser, 2002).

These shortcomings led to the creation of a strain dependent measurement of brittleness by Hajiabdolmajid (2001). This author considered the fact that during brittle failure, as cracks accumulate, the system’s cohesion must be decreasing as bonds break while the frictional component must begin mobilizing as cracks begin to coalesce and shear displacement occurs along them. This concept is presented in Figure 2-29, where the magnitude of residual strength as a function of shear strain is denoted as $\varepsilon_f^p$ and $\varepsilon_c^p$ for friction and cohesion, respectively. This led to a re-shaping of the brittleness index to the following form:

$$I_{Be} = \frac{\varepsilon_f^p - \varepsilon_c^p}{\varepsilon_c^p}$$

(2-28)
This new form allows for the consideration of how each component of strength is lost (cohesion) or mobilized (friction) as a function of crack accumulation and thus non-elastic strain. In terms of the physical system, this index demonstrates the volume of induced cracking and their relative ability to propagate within the sample (Hajiabdolmajid and Kaiser, 2002). This resolves the shortcomings shown in Eq.(2-27) by presenting a measure of stress path-dependent damage in the pre-peak region. For various plastic strain limits, the resulting rheological behaviour is shown in Figure 2-30, with A to E becoming increasingly ductile.

From more readily available lab testing values, Cai (2010) demonstrated that the Hoek-Brown parameter for strength, $m_i$, is approximately the ratio between compressive and tensile strength. As a result, it was also stated that it can be used as an indicator of the rock brittleness. This is intuitively satisfying as materials with low compressive strength would be thought to fail preferentially through shear before building up enough resistance to fail as a result of tensile induced cracking. According to this theory, for a rock to be considered brittle, it should have a minimum associated strength ratio of 15-20 although in some cases it has been shown that brittle mechanisms can occur at ratios as low as 10 (Cai, 2010).
Figure 2-30 – Demonstration of how the brittleness index ($I_{BE}$) considers the pre-peak inelastic behaviour in rock. This shows the stress path-dependent nature of strength loss and mobilization in rock (Hajiabdolmajid and Kaiser, 2002).

2.6.2 Intact- Laboratory Testing

Much of the understanding of brittle failure mechanics has been developed through laboratory testing. This has been accomplished using not only physical tests but also through numerical simulations using bonded particle modelling (Diederichs, 2003). Generally, the process of brittle failure is described based on the condition of crack interaction within the sample. These characteristic stress levels are shown in Figure 2-31, and include:

- Crack Closure (CC)- The closure of existing flaws and pore space in the rock as a result of initial loading conditions.

- Crack Initiation (CI)- The onset of inter- and intra-granular cracking as a function of heterogeneity and the volume of existing natural flaws in the rock. Cracks exploit the flaws to form and propagate in the loading direction. At the excavation scale, $CI$ is approximately the lower bound strength. It has been shown that this threshold relative to the ultimate sample $UCS$
can range from 20% in granitic rocks (Read, 2004), up to 60% in more granular rocks (Cai, 2010).

- Crack Damage (\(CD\))- This point marks the onset of interaction between previously isolated cracks in the sample. After this point, unstable crack propagation occurs, forming a defined shear band, until the ultimate strength of the sample is reached (Diederichs, 2007).

![Figure 2-31 – Stages of damage in brittle rock, demonstrating key thresholds of crack formation (after Diederichs 2003).](image)

2.6.2.1 Brittle Confinement Dependency

From work by Diederichs (1999) using bonded particle modeling (PFC), it was shown that the nature of macroscopic damage in brittle rock is highly dependent on the ratios of principal stresses- Figure 2-32. As it applies to lab scale specimens the following can be said:

- \(\sigma_3 < 0\) MPa (Tension)- rock splits in a rapid manner, with a roughly planar surface perpendicular to the applied load (assuming isotropic material). Once cracking is initiated in a tensile stress field, cracks propagate rapidly and in an unstable fashion (Cai, 2010).
• \( \sigma_3 = 0 \text{ MPa (Unconfined)} \)- samples fail through extensile axial fracturing which coalesce to form a high angle shear band- Figure 2-32b.

• \( \sigma_3 > 0 \text{ MPa (Confined)} \)- as confining stress increases, the formation of axial fractures becomes less pronounced, causing a lower angle shear zone. At extremely high confining stress, failure is truly through a shear dominated mechanism- Figure 2-32c/d.

![Figure 2-32 – Confinement dependency on cracking using a bonded particle model, showing; a) disk geometry, b) unconfined test , c) \( \sigma_3 = 20 \text{ MPa} \), and d) \( \sigma_3 = 60 \text{ MPa} \) (Diederichs, 2003).](image)

From work by several authors, it has been shown that there is relatively little confinement dependency of the crack initiation threshold. From work by Martin (1997) on Lac du Bonnet Granite, a constant deviatoric stress criterion for \( CI \) was proposed. This took the form of:

\[
CI = CI_{unconfined} + B \cdot \sigma_3
\]  

(2-29)

Reasonable values of \( B \) were suggested to be between 1.5-2.5, as confirmed by assuming typical inter-grain friction angles (Diederichs 1999, 2007). From testing of laboratory samples through cyclical loading, it was however shown that the crack damage threshold is quite sensitive to the level of confinement (Martin, 1993). As a result of this, it was concluded that for a rock experiencing an extremely prolonged loading cycle, the ultimate strength of the rock should gradually approach \( CI \). This
drop in $CD$ with load duration is less significant at higher confining stresses, as crack propagation is less pronounced in these stress states (Diederichs, 2007).

2.6.3 Excavation Scale Failure
The occurrence of brittle type failure in an underground setting is often depicted as spalling, or rock bursting. When in situ stress ratios are not at unity, the shape of failure is often characterized as “notch” shaped throughout the progression of brittle failure. This behaviour appears as excavation-parallel macroscopic fractures, which are stress induced and often seen in deep, lightly fractured ground conditions, as shown in Figure 2-33 (Martin, 1997). As a general criteria, Carter et al. (2008) postulated that the occurrence of brittle type failure can only occur in rockmasses with a minimum GSI of 60, and appropriate intact properties, as discussed in Section 2.6.1. In the case of strain bursting, the system must not only behave in a brittle manner, but also exhibit a significant dilational component (Diederichs, 2007). This effectively subjects the zone comprising the failed rock to high amounts of instantaneous convergence through a rapid and violent release of energy.

Figure 2-33 – Excavation parallel slabbing that occurs during spalling failure (after Read 2004).

In highly stressed openings and pillars, cracking begins because of the removal of confining stress (through excavation) and subsequent loading from the concentration of tangential stress. The initiation of this has been shown when $CI$ is exceeded by this critical stress concentration. Following this,
spalling will occur when stress exceeds the spalling limit, which is defined by the ratio of principal stress magnitudes. This relationship is rock type dependent, with typical heterogeneous materials requiring a $\sigma_1/\sigma_3$ ratio of $<10$ with more homogeneous rocks requiring a ratio of $>10$. This is shown in Figure 2-34, where at low confinements, the failure envelope is defined by the damage threshold-spalling limit (Diederichs, 2007). As confinement increases, the strength envelope is seen to greatly increase as the mode of failure becomes shear dominated. This portion of the envelope is defined by the crack damage threshold which has been shown at moderate to high levels of confinement to be the initiation point of yielding, as discussed in Section 2.6.2.1 (Diederichs, 1999, 2007).

![Figure 2-34 – Demonstration of the brittle failure envelope in principal stress space, with the respective failure mechanism types including; unraveling, spalling and shear dominated failure (Diederichs, 2003).](image)

### 2.6.4 Case Study- AECL’s Mine-by-Experiment

In the early 1990’s, Atomic Energy of Canada Limited (AECL) completed testing at the URL in Pinawa, Manitoba. Within the 3.5 m diameter tunnel in almost structureless granite, brittle behaviour was
monitored where displacement, strain, changes in stress and the occurrence of micro-seismic emissions were recorded. For the use in back-analysis of excavation damage through numerical modelling, the observed in situ damage profile from the tunnel in Figure 2-35 was used.

Figure 2-35 – Failed geometry around the URL tunnel, as defined by micro-seismic events, acoustic emissions and observed failure. Note the floor exhibited much less failure than the roof as a result of the failed material providing passive confinement (Read, 2004).

Hajiabdolmajid et al. (2002) used this profile in an attempt to replicate the extent of failure using various constitutive models. A summary of each is as follows, with the results shown in Figure 2-36:

- Elastic model- By completing an elastic stress analysis and comparing the induced stress surrounding the excavation with the rockmass strength, the extent of failure was evaluated. This significantly underestimated both the depth and extent of overbreak, as shown in Figure 2-36a. Read (1994) and Martin (1997) tried to overcome this limited result by successively removing any failed elements in the model until equilibrium was met. In both cases, this over predicted the size of failure by 2-3 times.
- Elastic-perfectly plastic model- This allowed for the consideration of plastic strain and subsequent stress redistribution on failure geometry. This was said to overestimate the strength of a brittle material as the inherent loss of cohesion prior to the ultimate plastic state is ignored. Although the result better predicts the failure zone as compared to the elastic model, it still greatly underestimated the depth of failure- Figure 2-36b. Hajiabdolmajid et al. (2002) regarded this modelling approach as the minimum depth of failure resulting from stress redistribution.

- Elastic-brittle model- Was first proposed by Hoek et al. (1995), in an attempt to capture brittle behaviour. The spalling process is imitated by decreasing both the Hoek-Brown parameters m and s, to near zero values within the post-peak region. This effectively acted to rapidly decrease cohesion to 20% of its peak value while also reducing the friction angle. As shown in Figure 2-36c, this still underestimated the depth of failure while now overestimating the lateral extent of failure.

- Cohesion-weakening-friction strengthening model- was first proposed by Hajiabdolmajid (2001), to address the shortcomings of the previously mentioned methods. The basis of this constitutive model is the reduction in cohesional strength and mobilization of friction with increasing plastic strains (Figure 2-37). As previously discussed, this behaviour closely mimics what is observed in laboratory scale tests of brittle material during subsequent crack accumulation and coalescence. By iteratively adjusting the plastic strain magnitudes at which cohesion and friction reach their final state, a close match could be made between what was observed in the URL with what was predicted through modelling (Figure 2-36d).
Figure 2-36 – Results of modeling from back-analysis of excavation damage at AECL’s URL using, a) fully elastic model, b) elastic- perfectly-plastic model, c) elastic-brittle plastic model and, d) CWFS model (after Hajiabdolmajid et al. 2002).

Figure 2-37 – Demonstration of cohesion loss and friction mobilization as a function of plastic strain accumulation, used in the CWFS constitutive model (Hajiabdolmajid, Kaiser and Martin, 2002).
2.7 Numerical Modelling Theory

Analytical solutions are valuable in the field of geomechanics as they provide quick and efficient results which also emphasize the most important required variables. The simplifications associated with most analytical methods are what also limit their application, as their use needs to be within the set of assumptions from the development of each method. These stipulations could include elastic behaviour, homogeneity, rockmass isotropy, time-independent responses or a static loading state (Bobet, 2010). Given the known complexity of geomechanical behaviour, often times analytical solutions are unable to capture the true solution. Analytical solutions almost exclusively also assume an idealized geometry of the problem, when in reality, many problems are dictated by the design geometry. It’s in these cases where solutions must be obtained numerically.

An important factor in numerical methods is that the result of the model is only as accurate as the inputs that are used. This is where experience must come in when using numerical methods when conceptualizing the problem. It’s these factors which ultimately drive the result of the model and must be carefully considered. Such aspects can include the chosen constitutive model and associated parameters which should ultimately come from some form of lab testing or in situ monitoring. It’s at this stage where, if the incorrect behaviour is chosen to represent the material in question, the model can be rendered useless, ultimately making the solution potentially detrimental to the chosen engineering design.

The interpretation of model outputs and the ability to relate them to real world behaviour is also critical to effective numerical modelling. It’s at this point of the process where experience of the modeler are important to produce a successful result (Carter et al., 2000; Jing and Hudson, 2002; Bobet, 2010).

Numerical models operate by dividing the model into discrete components which act to limit the system to a finite set of degrees of freedom. Each portion of the model must satisfy its own set of differential equations while also preserving continuity with the neighboring domains (Jing, 2003).

Although some of the governing equations to each numerical analysis type will be covered in this section, the focus is predominantly on the premise, applicability and limitations of each method.
In the broadest sense, numerical models can be classified as either continuum or discontinuum based (Jing and Hudson, 2002; Jing, 2003). Within each of these categories are a group of different solution methods, which are always increasing in complexity with the advent of a continued increase in computing power. Given the focus of the research using continuum based methods, particular emphasis will be placed on this throughout the following sections. Hybrid modelling methods such as FEM/DEM will not be reviewed given the lack of usage throughout this body of work.

2.7.1 Applicability of Model Type

The first question which must be asked during the modelling process is what method of representing the geomechanical system is the most appropriate; a continuum or discontinuum approach. The factor which drives this decision relates to the scale and importance of the discontinuities with respect to the entire model (Bobet, 2010). This has been eloquently summarized by Hoek (2006), where on one extreme you have a small scale intact sample containing no macro discontinuities while at the largest scale you may have a well-developed discrete fracture network, which must be accounted for (Figure 2-38).

![Figure 2-38 – Representation of the effect of sample size on discontinuity frequency as applied to an underground excavation and open pit slope geometry (after Hoek 2006).](image-url)
Brady (1987) provided some qualitative guidance on when either method may be the most appropriate. This is with the understanding of the capability of continuum models to either explicitly or implicitly represent discontinuities within the medium. In Figure 2-39a, a tunnel containing no structure is illustrated, thus a continuum based approach is what should be employed. Figure 2-39b shows a small number of critical discontinuities which slice the medium into a small number of continuous regions. Displacement in these areas will be continuous, however are discontinuous across the features. As a result, if a continuum model is used it must be able to account for these structures, particularly if they are thought to greatly impact the model results. In the case where the discontinuities show some continuity and repeatability, as is the case depicted in Figure 2-39c, the discontinuum approach should be elected to capture the displacement and rotation of each block within the system. If joint frequency is such that the blocks bounded by each discontinuity are much smaller than the excavation, then a pseudo-continuum approach can be employed (Figure 2-39d). In this case jointing can be implicitly reflected in the medium’s associated strength (Brady, 1987).

Figure 2-39 – Different degrees of rockmass structure and the proposed representation in numerical models (Bobet, 2010).
Ultimately the choice between a continua and discontinua approach is based on the ability of either selection to approximate the observed material behaviour. Despite this important distinction, continuum based models are far more common in practical rock engineering— even if it is not the most appropriate. The driver of this is purely based on the complexity of discontinuum models and the inherent expertise required to properly develop these models. Despite this, there is an increasing recognition within the rock mechanics community of the shortcomings of continuum modelling in certain geological conditions. Such cases include the macroscopic separation of blocks under high stress or the analysis of gravity driven wedge failures (Bobet, 2010).

2.7.2 Continuum Methods
Within the realm of continuum based methods, the three predominant types in geomechanics include; the Finite Element Method (FEM), the Finite Difference Method (FDM) and the Boundary Element Method (BEM). It’s with the use of these methods which the first modelling software in the late 1970’s and 1980’s was based on. In this section only FEM and FDM are considered as they represent the modelling techniques applied throughout the research.

2.7.2.1 Finite Element Method
Since its conception, the Finite Element Method has emerged as the most prominent form of numerical modelling across multiple engineering disciplines. The method is based on the discretization of the continuum into smaller elements which all intersect at their respective nodes (Figure 2-40). With the use of appropriately chosen interpolation functions, the displacements within each element are determined based on the displacement of the nodes which surround that element in question, which must always balance for the model to be in equilibrium.
2.7.2.1.1 Theory

The general procedure taken when implementing the FEM is as follows:

1) Discretize the problem domain
2) Define how the unknown varies inside each element
3) Develop the stiffness matrix for each element
4) Assemble the global stiffness matrix
5) Solve the equations of equilibrium

Once the model geometry has been defined, the continuum is discretized into finite elements which take on a simplified geometry such as a tetrahedron. Despite this process often being highly automated in most software packages, the chosen discretization density has the potential to greatly impact the results of the model. This is particularly the case in highly brittle materials. If element size is too coarse, this fails to capture the material behaviour with enough detail, while a mesh that is too fine can create artificial nodal effects.

The variables that define the condition of an element are not stored within that element, rather they are found at each of the nodes which confine the element. This then makes it important how the
variables vary between each node to reflect the average state of each element. In a purely two-
dimensional model a shape function is used to define how the variables change surrounding the element. The following is the generalized form of the shape function:

\[ q(\xi, \eta) = \sum_n N_n (\xi, \eta) \cdot q_n^e \]  

(2-30)

where \( q \) is the quantity of interest located at the point defined by \( \xi, \eta \). The shape function itself is \( N_n (\xi, \eta) \) at the node \( n \). The value of the quantity of interest is \( q_n^e \) at node \( n \) and element \( e \). In Phase\(^2\) the simplest shape function is that for a triangular element, where each quantity varies in a linear manner in every direction, and the value in each element is the average of each nodal quantity surrounding the element, weighted to the location of each node relative to the point in question.

Potentially the most important part in the FEM is the theory of the development of the stiffness matrix of an individual element for the purpose of mapping it to the global system. This process begins by first defining how the displacements relate to the strains inside each element. This relationship is shown by matrix, \( B^e \). Since the displacements in an element inherently are dictated by the calculated values at each node (through the shape function) and strain is a partial derivative of the displacement, \( B^e \) is therefore a matrix comprising partials of the shape function, \( N_n(\xi, \eta) \). The matrix size is the number of nodes surrounding the element multiplied by the number of partial derivatives. Sticking with the 2D analysis, if each element is triangular with three defining nodes, then the matrix will be 3x6 as there are two partials which define each node. If we allow \( u^e \) to be a vector denoting all nodal displacements around an element, \( e \), then the strain in an element can be defined as:

\[ \varepsilon = B^e u^e \]  

(2-31)

To fully define the stiffness matrix, the constitutive matrix must be used, which relates stresses to strains. For an elastic and isotropic medium in plane-strain conditions, the following matrix, \( D \) can be used in 2D:
\[ D = \frac{E}{(1 - \nu)(1 - 2\nu)} \begin{bmatrix} 1 - \nu & \nu & 0 \\ 0 & 1 - \nu & 0 \\ 0 & 0 & \frac{1}{2}(1 - 2\nu) \end{bmatrix} \]  

(2-32)

This then allows the stiffness matrix for a given element to be defined by:

\[ K^e = \int B^e DB^e dV \]  

(2-33)

where \( K^e \) is a larger matrix which is filled with submatrices. For a triangular element denoted as \( e \) this is represented as:

\[ K^e_1 = \begin{bmatrix} k_{11} & k_{12} & k_{13} \\ k_{21} & k_{22} & k_{23} \\ k_{31} & k_{32} & k_{33} \end{bmatrix}^{e_1} \]  

(2-34)

where each component, \( k_{ij} \), is a 2 x 2 matrix defined by the matrices \( B_i \) and \( B_j \) which are submatrices of the matrix \( B^e \). The sub-components of Eq. (2-1) are then shown as:

\[ K_{ij} = B_i^T DB_j At \]  

(2-35)

To place each individual element matrix into the context of the global stiffness matrix, this is done simply by the addition of each matrix, \( K^e \). This is written as:

\[ K = \sum_n K^{en} \]  

(2-36)

Once the global stiffness matrix has been fully defined, the equilibrium equation must then be solved. This is dictated by the minimum potential energy principle and is given by:

\[ Ku = F \]  

(2-2)

where \( K \) is the global stiffness matrix, \( u \) is the nodal displacement vector and \( F \) is the vector of forces applied to each node.

This can be solved for based on the defined displacement conditions at the model boundary using a method such as Gaussian elimination. If the material constitutive model considers plastic behaviour, the stiffness matrix then has a dependency on the displacements which forces Eq. (2-2) to be solved incrementally using a time step.
Although this summary of mathematical principles behind the FEM provides a basic idea, for further information, the reader should be directed to the two main references from this section by Owen & Hinton (1980) and Beer & Watson (1992).

2.7.2.1.2 Application and Limitation

The general consensus throughout the geomechanics industry is that FEM represents the most popular type of analysis for design purposes. The strengths of using codes such as Phase2 include as follows:

- FEM can allow for the capture of staged models throughout the evolution of “pseudo-model time”. This is valuable when considering staging of excavation design which can also include the necessary staged support strategy.

- Codes can handle certain degrees of material complexity such as heterogeneity and anisotropy, giving the user flexibility. Heterogeneity must be explicitly defined by adding zones of distinct materials in the model while anisotropy can be simply modelled by altering the stiffness matrix of the elements. This is generally done through changing the definition of the constitutive matrix.

- FEM codes represent some of the oldest modelling techniques within the geotechnical industry and therefore a great deal of experience throughout a large community exists (Carter et al., 2000; Jing, 2003).

With these strengths in consideration, there also includes some significant disadvantages to FEM. The most prominent of these include:

- From the description of simple mathematics behind FEM described above, the reader can intuitively see that the calculations involve large systems of matrices which have a greater degree of memory required as compared to other modelling methods.

- As previously mentioned, the way the model is discretized can create great problems with regards to model response. This is a greater issue in certain material types as compared to others (ex. brittle rock). This mesh dependency may not only affect results but also can greatly increase the computational intensity during each run.
• Although an elastic model in FEM is relatively straightforward mathematically, when trying to consider more complex material behaviour such as strain-softening, this requires far more intricate algorithms to ensure model equilibrium.

• To date, implementing discrete fractures is quite difficult. This has resulted in large bodies of work in proposing methods to introduce joint elements into FEM. Currently, the best way of implementing it is by adding an element of zero thickness which is defined by four nodes. The joint is given a shear and normal stiffness, however given the methodologies of FEM, this only allows for small scale displacements to be captured. This in turn doesn’t allow the model to analyze the occurrence of sliding along joints or the detachment of blocks along joint contacts. This inhibits finite element models from accessing fracture growth, which has been identified by Jing (2003), to be FEM’s greatest setback.

2.7.2.2 Finite Difference Method
The use of finite difference method (FDM) in numerical modelling operates under the premise that the equations which govern the model behaviour can be represented using finite differences—hence the name. This method has been in existence for the longest of all numerical methods in geomechanics and predates usage of the modern computer (Bobet, 2010). The first recorded usage of the method was by Runge in 1908 who used it to solve torsion problems (Timoshenko & Goodier 1970). By implementing the finite differences to the set of differential equations, this allows the entire system to be reduced to a group of linear equations, which in turn allows the problem to be solved using any classical method. Given this simplification, and with the occurrence of computer usage in geomechanics, has allowed the use of FDM extensively. The most common software packages employing FDM are FLAC and FLAC$^3$D by Itasca Consulting Group Inc.

Somewhat similar to the FEM, the finite difference method places a grid over the domain that is being modelled (Figure 2-41). In this case, each point is given coordinates which have a relative location to all other points in the model. Discontinuities can also be represented by adding other grid points on
each side of the fracture. The displacement between the grid point pairs then determines the slip and normal separation along the fracture. These fractures can also behave according to frictional theory by defining a constitutive equation such as Mohr-Coulomb to define the contained grid points. When discontinuities are considered, the modelling type takes on more of a “hybrid” approach.

![Discretization using the Finite Difference Method in 2D](image)

**Figure 2-41 – Discretization using the Finite Difference Method in 2D (Bobet, 2010).**

2.7.2.2.1 Mathematical Theory

Given the extensive usage of Itasca’s FDM codes throughout this body of work, the theory described in this section will be based on the formulations of FLAC. Although FLAC\textsuperscript{3D} will not be described, the theory is relatively similar to that of its 2D counterpart. For further reference on the content of this section refer to Itasca’s manuals for FLAC (Itasca Consulting Group, 2011).

As FLAC is an explicit solution, an arbitrary time-step is used to progress through the necessary states of displacement and force based on the constitutive properties of the model. The most notable benefit of this is that the equations which govern motion in the model will remain stable even if the model itself becomes unstable. This then allows these codes to consider dynamic applications such as seismicity (Itasca Consulting Group, 2011).
The process of solving through each time-step is shown in Figure 2-42, which begins using Newton’s second law, and dictates how the model stresses and forces are translated into velocities and displacements throughout each step. This then uses the constitutive relations to work through what stress changes and strain accumulations must have occurred during the time-step. This is repeated continuously during every time step until the model reaches a solution. The most important part of this methodology is that for subsequent calculation steps, the model state is assumed to be constant. Although across large time-steps this is problematic, by making the time steps so small in FLAC, any variation in stress that has occurred physically can’t propagate to other zones within the given step.

During the solution process in FLAC, the most general equation of motion is as follows:

\[ \rho \frac{\partial v_i}{\partial t} = \frac{\partial \sigma_{ij}}{\partial x_j} + \rho g_i \]  \hspace{1cm} (2-37)

where \( v_i \) is the velocity, \( x_j \) is the direction, \( \sigma_{ij} \) is the component of the stress tensor, and \( g_i \) is used to consider the occurrence of body forces from the weight of the other zones.

When motion is calculated in FLAC, this is in terms of forces and displacements occurring at each node. The sum of all forces at a node can then be used to calculate new acceleration at that point, while new velocities can be determined based on the previous velocities. It is this consideration of the previous step which makes it a finite difference solution. This allows for the update of the stresses which

![Figure 2-42 – Workflow used during FDM modelling in FLAC (Itasca Consulting Group, 2011).](image)
follows a series of steps that begin with the consideration of stress change due to the zone rotation followed by the stress adjustment based on the constitutive relation. This then allows us to re-write the previous Eq.(2-37) in terms of finite differences, which considers the time, $t$, of each step:

$$v_{i}^{t+\Delta t/2} = v_{i}^{t-\Delta t/2} + \sum F_{i}^{t} \frac{\Delta t}{m}$$  \hspace{1cm} (2-38)

where $F_{i}^{t}$ is the sum of forces at a particular node and $v_{i}^{t-\Delta t/2}$ are the velocities calculated at time, $t$. By knowing the new velocities, the displacements, $u_{i}$, strains, $\varepsilon_{ij}$, and rotation, $\omega_{ij}$, can also be further updated according to:

$$u_{i}^{t+\Delta t/2} = u_{i}^{t} + v_{i}^{t+\Delta t/2} \Delta t$$  \hspace{1cm} (2-39)

$$\varepsilon_{ij} = \frac{1}{2} \left( \frac{\partial v_{i}}{\partial x_{j}} + \frac{\partial v_{j}}{\partial x_{i}} \right)$$  \hspace{1cm} (2-40)

$$\omega_{ij} = \frac{1}{2} \left( \frac{\partial v_{i}}{\partial x_{j}} - \frac{\partial v_{j}}{\partial x_{i}} \right)$$  \hspace{1cm} (2-41)

This allows for the update of the stresses which follows a series of steps that begin with the consideration of stress change due to the zone rotation followed by the stress adjustment based on the constitutive relation. These are represented as:

$$\sigma_{ij}^{t+\Delta t} = \sigma_{ij}^{t} + (\omega_{ik}\sigma_{kj} - \sigma_{ik}\omega_{kj}) \Delta t$$  \hspace{1cm} (2-42)

$$\Delta \sigma_{ij} = C_{ijkl}\Delta \varepsilon_{kl}$$  \hspace{1cm} (2-43)

where $C$ is the constitutive matrix that defines the material behaviour based on input of the modeler. The change in stress, $\Delta \sigma_{ij}$, is then applied to the existing model stress state.

The final step in the process described in Figure 2-42, is to determine the nodal forces within the model for the given time-step. This essentially represents the state of disequilibrium which exists and must be reduced in order for the model to reach a solution. This force, $F_{i}$, is shown as:

$$F_{i} = \frac{1}{4} \sum \sigma_{ij} n_{j} S$$  \hspace{1cm} (2-44)
where the force is the sum over the 4 triangular elements which encompass a node, $n_j$ are the normal to each side of the quadrilateral which are in contact with the node and $S$ is the side length of each portion of the triangular elements.

The calculation steps described above continue through every time-step until the model reaches a state of equilibrium as defined by the portion of remaining unbalanced forces, shown in Eq. (2-44). This process is further aided by the use of damping when trying to arrive at static solutions. If this wasn’t used, model inertia would preferentially create dynamic motion, which wouldn’t converge to a model solution (Cundall, 1982). This is done by applying a viscous damping force at each node and is related to the rate of change in kinetic energy within the system.

2.7.2.2.2 Application and Limitation

Given the continuum basis of both FDM and FEM, the main differences between them may not be the most apparent. The list below acts as a contrast between the two methods to provide some insight into when one is more applicable than the other. The strengths of FDM compared with FEM include, but are not limited to:

- By solving through the use of an explicit solution, it avoids having to create large matrices
- The analysis of plasticity is formed by still solving the same equations as for an elastic analysis, unlike in FEM where higher complexity algorithms are required to accomplish this.
- The solution path of FDM allows for the consideration of dynamic problems
- Models can observe and quantify large strains

Some notable disadvantages of FDM as compared to FEM include:

- The inefficiency of the code at evaluating simple linearly elastic problems
- In the case of static problems, reaching a solution is dependent on the parameters used in the damping function and also the user’s ability to judge the adequacy of model convergence.

For further reference on the content of this section or to review the mathematic principles behind finite difference solutions, refer to Itasca’s manuals for FLAC (Itasca Consulting Group, 2011).
Chapter 3

Laboratory Testing and Rockmass Characterization

3.1 Introduction

Data from two boreholes at KGHM’s Victoria Project were used in the analysis presented in this dissertation. The holes provided information for geomechanical characterization and design of the production and ventilation shafts, which have targeted development depths of 1850 mbgs. It is approximately along the axis of these shafts where the vertical boreholes were completed. This includes borehole FNX-1204, which was finished in 2011 prior to a shaft realignment, which dictated the creation of the second borehole, GT0020VCa, in 2016.

Each borehole includes complete geological and geotechnical logging which was done during drilling by KGHM staff and various external consultants. In addition to this, acoustic teviewer surveys were used to supplement the structural core logs and to assess the frequency of borehole breakout. In the more recent hole, a hydro-fracturing program was completed, although no usable results were obtained from this work. Given that borehole GT0020VCa was completed throughout the time of this thesis work, it was solely used for laboratory testing and is the focus of much of the subsequent in situ stress and overbreak characterization work in the following chapters.

3.2 Investigated Lithologies

As discussed in Chapter 2, the host-rock geology surrounding the Victoria Project consists of steeply dipping interlayered metasediments and metavolcanics of the Huronian Supergroup, which have subsequently been intruded by later stage mafic intrusions (Farrow et al., 2011). In Figure 3-1, the generalized geological model can be seen with the approximate locations of the shafts and the ore body.
Figure 3-1 – View of KGHM’s Victoria Project with sulphide mineralization shown in red (Zones 1, 2, 4, Mini) and shaft infrastructure shown in yellow. The prominent Creighton Fault to the North of the deposit is represented as the blue plane (courtesy of KGHM).

3.2.1 Lithological Description

The following description of the intersected geologies is an excerpt from the NI 43-101 report completed for the Victoria Project by Farrow et al. (2011). Figures of each lithology were taken during the geological logging process by KGHM geologists and further supplemented by photographs taken during sample collection for this thesis.

3.2.1.1 Metasedimentary Rocks (MTSD/QTZT)

The metasedimentary rocks are part of the Stobie Formation and range in composition from quartzite (QTZT) and arkosic quartzite to grey argillaceous arkose (Figure 3-2). They have thick (10 to 20 m),
steeply dipping beds. Locally the argillaceous arkose component is intensely foliated and contains variable amounts of clastic quartz, feldspar, biotite, epidote and sericite. Pyrrhotite is a common sulphide observed locally within the pelitic units as thin layers along the foliations. What have been interpreted as tabular aluminosilicate porphyroblasts have been observed throughout the pelitic sediments but have not been mineralogically characterized. The metasedimentary rocks are commonly interlayered with other footwall lithologies and appear to be fairly continuous along strike and dip.

![Core photo](image)

**Figure 3-2** – Core photo taken from pilot hole GT0020VCa from 1084 to 1095 m depth. Upper runs of core show the more mafic MTSD unit with the sharp contact with the QTZT unit.

3.2.1.2 Metabasaltic Rocks (MTBS)

The metabasalts in the vicinity of the boreholes are part of the Stobie Formation. In both cases they range in colour from light grey-green to dark green-black and consist of fine- to medium-grained amphibolite or amphibolite schist (Figure 3-3). These mafic to intermediate, submarine metavolcanic rocks are weakly foliated and occur as flows containing amygdules and small phenocrysts, with the development of pillows locally. They consist of hornblende, actinolite, plagioclase, quartz, chlorite, epidote, pyrite, pyrrhotite, magnetite and ilmenite. To the south of the Creighton Fault the steeply dipping metabasalt occurs as
flows that are 75 to 300 feet (23 to 91 m) in thickness that are commonly interlayered with
metasedimentary and metagabbroic units. Minor late quartz veining is observed with associated pervasive
chlorite alteration.

Figure 3-3 – Core photo taken from pilot hole GT0020VCa from 1151 to 1163 m depth showing the
MTBS unit with a 0.5 m quartz vein.

3.2.1.3 Metagabbro (MTGB) and Metacrystic gabbro (MXGB)
The metagabbro and metacrystic gabbro are similar rock types that mainly differ in terms of their grain
size. These green to dark green rocks are composed of amphibole, plagioclase, chlorite, epidote, quartz,
and biotite (Figure 3-4). The metacrystic gabbro consists of very coarse-grained, optically continuous,
poikiolitic crystals of amphiboles, whereas the metagabbros are fine to medium grained. The metacrystic
gabbro may be a re-crystallized version of the metagabbro. Both the metagabbro and metacrystic gabbro
are steeply dipping, and are interlayered within the metasedimentary and metabasaltic rocks (perhaps
having been intruded as sills). Minor interstitial pyrrhotite-pentlandite and chalcopyrite is locally
associated with some units of the metagabbro (Nippising Gabbro), but predates the Sudbury Event
mineralization. Minor quartz-carbonate veining occurs with associated chloritic alteration halos.
Figure 3-4 – Core photo taken from pilot hole GT0020VCa from 895.5 to 907 m depth. Upper runs of core show the coarser grained MXGB unit with a gradational contact to the MTGB.

An example of the gradational change between the metagabbro and metacrystic gabbro end members, can be seen in Figure 3-5. In the left-most image the grain size is less than 1 mm and can be described as aphanitic. This progresses to a grain size in the 1-2 mm range (Figure 3-5b) and then to the most coarsely grained sample that was collected, with a grain size of the 3-6 mm (Figure 3-5c).

Figure 3-5 – Example of gradational change between metagabbro and metacrystic gabbro end members, as denoted by increase in grain size from a) BO-MTGB-915.3-7, b) BO-MTGB-902.5-2 and, c) BO-MTGB-909.1-4.
3.2.1.4 Rhyolite (RHY)

The rhyolitic units at The Victoria Project are typically fine-grained, light green to grey in colour, and commonly display white amygdules within the groundmass (Figure 3-6). Fragmental rhyolite units are also observed locally within the massive rhyolite. The rhyolite is commonly siliceous, and the groundmass consists of anhedral quartz, with less common amphibole, biotite, chlorite, and epidote. Minor quartz veins are ubiquitous in the rhyolitic rocks.

![Core photo](image)

**Figure 3-6 – Core photo taken from pilot hole GT0020VCa from 1594 to 1610 m depth. RHY is seen throughout although a distinct textural difference is obvious throughout each run.**

During the core logging process, the degree of anisotropy in the rhyolite appeared to increase with depth in the hole. This could be indicative of multiple extrusions of the protolith rock that had slightly different chemical signatures between each or localized intensity in ductile strain. This has created what is shown in Figure 3-7, where the most shallow sample has only a faint foliation with very little amygdules. This increases in Figure 3-7b, where the amount of amygdules has sharply increased and appear to be preferentially aligned along a more developed foliation. The final sample is from approximately 1700 m depth, where a strongly developed foliation is denoted by lenses of dark minerals. Upon inspecting the cross-section of this sample, it can be seen that the foliation is creating sharp
undulations along the surface. The implications of the various degrees of anisotropy observed in this rock type are further explored in Chapter 4, where the effects of this on breakout are modelled and discussed.

Figure 3-7 – Different degrees of anisotropy observed in the rhyolite lithology showing a) the most homogeneous sample (BO-RHY-962.1-9), b) faint foliation with preferentially aligned amygdules (BO-RHY-1499.2-48) and c) the most intensely foliation RHY, as defined by the black lenses (BO-RHY-1695.5-60).

3.2.1.5 Quartz Diabase Intrusions (QDIA)

The quartz diabase dykes are the oldest dyke rocks observed on the Victoria Project and consist of anhedral amphibole, quartz (< 5%), plagioclase, and trace titanite (Figure 3-8). Limited drilling has facilitated the interpretation that these dykes occur discontinuously in an east to west direction sub-
paralleling the Creighton Fault; they may also pinch and swell in the dip direction. The timing of emplacement of the quartz diabase dykes is poorly defined, but is considered to be pre- or pene-contemporaneous with the Sudbury Event.

![Core photo from pilot hole GT0020VCa from 1525 to 1542 m depth, containing QDIA.](image)

**Figure 3-8** – Core photo from pilot hole GT0020VCa from 1525 to 1542 m depth, containing QDIA.

### 3.3 Rockmass Evaluation
The characterization of rockmass conditions was completed in both pilot holes from conventional core logging methods and with the review of ATV surveys. These were both completed by external consultants, although the results from GT0020VCa were validated by the author during the sample collection process. In the case of the logging of borehole FNX-1204, Kalenchuk & Hume (2015) raised concerns regarding the lack of variability in the description of joint characteristics. In the case of both the joint roughness and joint alteration, over 98% of the raw data gave values of 1.5 and 0.75 respectively, for each parameter. As a result of this, only pilot hole GT0020VCa was used for the description of rockmass characteristics using the following methods:
where $Q'$ is the modified Norwegian Q-system developed by Barton et al. (1974), $RQD$ is the Rock Quality Designation (Deere et al., 1967), $J_n$ is the joint set number, $J_r$ is the joint surface roughness and $J_a$ is the joint alteration factor.

The geomechanics classification system known as the Rock Mass Rating (RMR) was also considered during logging. This was first developed by Bieniawski (1976) and further updated in 1989 which reflects its current state (Bieniawski, 1989). This takes the following form:

$$RMR^{1989} = A1 + A2 + A3 + A4 + A5$$

where $A1$ is the intact rock strength, $A2$ is the Rock Quality Designation-RQD, $A3$ is the spacing of discontinuities, $A4$ is the condition of discontinuities and $A5$ is the presence of groundwater. Given that groundwater conditions must be observed in situ, no value indicative of true rockmass conditions could be given to parameter $A5$. Knowing that at depth in Sudbury there is typically very little groundwater, dry conditions were assumed throughout the determination of the Rock Mass Rating (Zhang, 2016).

Despite the lack in confidence of the data obtained regarding the joint state in FNX-1204, the fracture frequency and $RQD$ were reviewed from core logging data and also from the ATV logs, which were used in each of the boreholes. This provided some form of quality control on the data, but also an alternative method for characterizing joint set occurrence and orientation. The comparison between the boreholes is discussed in detail in Section 3.3.2.

### 3.3.1 Rockmass Characterization- GT0020VCa

During core logging of pilot hole GT0020VCa, rockmass characterization parameters were collected for every 3.0 m length of core along the length of the borehole. As can be seen in Figure 3-9, there appears to be relatively little correlation between rock type and rockmass competency. This discord could potentially be explained by the similarity of material properties between lithologies, as described in Section 3.4.4. There is however somewhat of a relationship between the occurrence of major through-going
discontinuities, such as faults (FLT) and shear zones (SHZ), on the rockmass rating. This is particularly obvious near the bottom of the hole at 1850 m where a zone of steeply dipping faults causes significant interruption in rockmass continuity. From the Norwegian Q system this rock would vary from; Fair ($Q' = 4-10$) to Good ($Q' = 10-40$) quality without the consideration of groundwater and in-situ stress state. From the RMR system, the rock grades at; Good (60-80) to Very Good (80-100). It is also important to note that for the use in borehole breakout analysis, only the conditions from 800 m downwards are considered, as above this, no appreciable breakout was observed.

Figure 3-9 – Rockmass characterization of pilot hole GT0020VCa, considering Norwegian $Q'$ (left) and RMR$^{1989}$ (left). Black trendlines is the inverse distance weighted mean.
3.3.2 Borehole Comparison-Structural Frequency

The frequency of discontinuities in both boreholes was logged in consideration of the occurrence of both jointing, faulting and shearing. No through-going major structures were plotted in Figure 3-10, as they varied between each borehole. Despite this, it can generally be seen that there is an agreement of general discontinuity frequency along the borehole, with some increased fracturing at 200 m, 1000 m and 1800 m depth. This is reflected not only in the fracture frequency (FF), but also the RQD plot. One noticeable difference between the two boreholes is the fluctuation in values and the synchronicity of changes between the RQD and FF plots for each hole. In the case of GT0020VCa, where a decreased RQD is seen, this is usually reflected by an increase in FF per meter. This is highly intuitive, although it is not always seen in FNX-1204. Over the length of this hole, there generally is a lack of change in the interpreted fracture frequency, which further lends itself to the previous discussion on data quality issues with pilot hole FNX-1204.

In pilot hole GT0020VCa, there appears to be a sharp increase in fracture frequency at 950 m. This is somewhat reflected in a lower RQD for the remainder of the hole, but not to the same degree. This depth represents an important transition in the borehole as it was at this point where wedging was required to correct for borehole deviation and the bit size was changed from HQ to NQ. This also is roughly the point where the borehole began to incur some breakout and thus the beginning of overstressed rock conditions. These are all potential factors which may have led to the sharp increase in fracture frequency, to varying degrees. From the inverse distance weighted mean from each hole, RQD never was less than 80%, which corresponds to; Good (75%-90%) and Very Good (90%-100%) rock. This, along with the rockmass rating described in Section 3.3.1, agrees well with the observations made during sample collection, as shown in Figure 3-2 to Figure 3-8.

From the inspection of the comparison in fracture frequency between core logging and ATV surveys for both pilot holes, it would be expected that the FF during core logging should exceed that of the downhole logging technique. This relates to the process of removing the rock from in situ conditions, which could cause the opening of joints within the core boxes that may have been too tight in the ground
to be observed by the acoustic teviewer. During core handling and transportation, this should also cause further disruption of the core that can lead to mechanical breaks, which in practice shouldn’t be logged as jointing, however this inevitably may occur over the entire length of the borehole. As can be seen in Appendix A Figure A-1, this is the case in the pilot hole GT0020VCa, where the fracture frequency from core logging was always greater than from the ATV survey. This however is the exact opposite trend in borehole FNX-1204, which further demonstrates some apparent lack in logging QA/QC.

Figure 3-10 – Comparison of fracture frequency (left) and RQD (right) between pilot holes FNX-1204 (a) and GT0020VCa (b). Trendlines are created using the inverse distance weighted mean approach.
3.3.3 Joint Set Interpretation

The orientation of discontinuities within each hole was acquired from borehole televiewer surveys, which can resolve a dip and dip direction of each fracture based on its trace around the borehole wall and the borehole orientation. An example of this is shown in Figure 3-11, where the lower most discontinuity has a corresponding orientation of $34^\circ/281^\circ$.

![Figure 3-11 – Example of structure from the ATV survey in pilot hole GT0020VCa. Structure trace of several joints around the borehole wall (left) is highlighted in purple with the interpreted structure orientation in a 3D log (right).](image)

The following Figure 3-12 shows the interpreted joint set trends from both pilot holes. A detailed orientation and further subdivision of joint sets based on rock type is given in Table 3-1. Across both boreholes, there are three prominent, moderately- to steeply-dipping joint sets with one minor, sub-horizontal set. With the exception of the metasediments, these are seen throughout all other rock types. This could potentially be due to the inherent difference in protolith rocks between the MTSD and all other lithologies or could be simply from the lack of joint measurements from this rock type. Within the rhyolitic unit, joint set B preferentially occurred within both pilot holes. Both the dip and dip direction of this set closely corresponds with the observed foliation in that unit, which is further described in Section
3.4.4. Given the proximity of the interpreted joint sets to the blind-zone (Figure 3-12-shaded region), it is likely that the actual dip of the joint sets is greater than what is listed, given the inability of an ATV survey to pick up vertical/sub-vertical fractures.

Figure 3-12 – Review of interpreted joint set orientations from ATV survey in pilot holes FNX-1204 (left) and GT0020VCa (right), showing three major and one minor joint set. Shaded region represents a 10° blind-spot. A full summary of structural trends from each lithology can be found in Table 3-1.
Table 3-1 – Summary of interpreted joint set orientations from ATV surveys in pilot holes FNX-1204 and GT0020VCa, with breakdown of structural trends based on lithology.

<table>
<thead>
<tr>
<th>Lithology</th>
<th>Structural Trend- Dip⁰/Dip Direction⁰ (+/- 2 std.dev)</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D-minor set</th>
</tr>
</thead>
<tbody>
<tr>
<td>FNX-1204</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>All</td>
<td>73/294 (23.6)</td>
<td>70/262 (14.5)</td>
<td>75/190 (18.9)</td>
<td>15/288 (26.2)</td>
<td></td>
</tr>
<tr>
<td>MTBS</td>
<td>68/302 (13.1)</td>
<td>71/267 (21.8)</td>
<td>72/224 (10.8)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MTGB</td>
<td>71/290 (19.0)</td>
<td>66/261 (13.2)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MTSD</td>
<td>75/301 (12.8)</td>
<td>61/196 (19.1)</td>
<td>21/308 (8.6)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>QDIA</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Insufficient Data</td>
</tr>
<tr>
<td>RHY</td>
<td>78/304 (19.7)</td>
<td>68/273 (10.1)</td>
<td>75/222 (13.1)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>GT0020VCa</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>All</td>
<td>56/288 (21.1)</td>
<td>66/246 (20.3)</td>
<td>70/190 (20.3)</td>
<td>13/285 (22.4)</td>
<td></td>
</tr>
<tr>
<td>MTBS</td>
<td>50/284 (14.6)</td>
<td>69/244 (8.9)</td>
<td>71/204 (17.1)</td>
<td>11/288 (15.8)</td>
<td></td>
</tr>
<tr>
<td>MTGB</td>
<td>61/292 (20.9)</td>
<td>62/251 (19.5)</td>
<td>59/187 (14.2)</td>
<td>29/282 (23.8)</td>
<td></td>
</tr>
<tr>
<td>MTSD</td>
<td></td>
<td>71/187 (19.5)</td>
<td>02/325 (21.2)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>QDIA</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Insufficient Data</td>
</tr>
<tr>
<td>RHY</td>
<td>67/304 (23.5)</td>
<td>69/245 (18.7)</td>
<td>77/211 (16.7)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

3.4 Intact Laboratory Testing

In the case of both pilot holes, laboratory testing was undertaken for all major lithologies to characterize intact rock strength and behaviour. In the case of the first hole, this involved the determination of peak unconfined compressive strength and intact rock stiffness. For the work completed on the more recent hole as part of this thesis, the crack accumulation thresholds from $UCS$ and $BTS$ were determined through the use of strain gauges and acoustic emission ($AE$) monitoring during testing.

3.4.1 Past Results- FNX-1204

Testing from pilot hole FNX-1204 was done by the Robert M. Buchan Department of Mining testing lab at Queen’s University. This involved a testing regime on 42 samples where the unconfined compressive strength and stiffness was evaluated using bracelet style strain measuring tools (Kalenchuk and Hume, 2015). As summarized in Table 3-2, the test results have been reported based on samples which failed through the intact material (termed as through the matrix) and those which failed along healed structures.
and pre-existing planes of weakness. This is an important consideration which was also evaluated during the more recent testing regime. Although the specific details about load rate and testing time weren’t immediately available, all other sample preparation parameters for the test are assumed to conform with the established ASTM standards for UCS testing (ASTM, 2013b). The comparison between results from both testing regimes are discussed and compared further in Section 3.4.5.4.

Table 3-2 – Summary of UCS testing results from pilot hole FNX-1204 (after Kalenchuk & Hume 2015).

<table>
<thead>
<tr>
<th>ROCK TYPE</th>
<th>Density (g/cm³)</th>
<th>UCS (MPa)</th>
<th>Young's Modulus (GPa)</th>
<th>Poisson's Rat.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Density</td>
<td>All Data</td>
<td>Matrix Only</td>
<td>All Data</td>
</tr>
<tr>
<td>MTSID</td>
<td>2.8</td>
<td>104.0</td>
<td>56.0</td>
<td>54%</td>
</tr>
<tr>
<td>QTZT</td>
<td>2.8</td>
<td>147.4</td>
<td>26.0</td>
<td>18%</td>
</tr>
<tr>
<td>MTBS</td>
<td>2.9</td>
<td>115.7</td>
<td>32.0</td>
<td>28%</td>
</tr>
<tr>
<td>MTGB</td>
<td>3.0</td>
<td>80.2</td>
<td>45.0</td>
<td>56%</td>
</tr>
<tr>
<td>MXGB</td>
<td>3.0</td>
<td>134.0</td>
<td>50.0</td>
<td>37%</td>
</tr>
<tr>
<td>RHY</td>
<td>2.7</td>
<td>145.4</td>
<td>30.0</td>
<td>21%</td>
</tr>
<tr>
<td>QDIA</td>
<td>3.0</td>
<td>142.7</td>
<td>137.0</td>
<td>96%</td>
</tr>
</tbody>
</table>

Avg. = Average
St. Dev = Standard Deviation
COV = Coefficient of Variation

As can be seen from the reported results, in all cases, the purely matrix derived failure showed a greater peak strength and stiffness. This should come as no surprise given the known influence of heterogeneity on the strength of geomaterials. This dependency was also shown in the coefficient of variation calculations which consistently showed that when failure occurred through intact material, the result was far more repeatable and indicative of homogenous rock conditions. Of particular concern are the values of Poisson’s ratio, which are far lower than the typically assumed 0.2-0.3 for practical applications in geomechanics (Gercek, 2007). Kalenchuk & Hume (2015) attributed this to the fact that the strain gauge bracelets were removed halfway through the testing cycle to protect them from damage upon sample failure. As a result, the body of data used to calculate the Poisson’s ratio may not be representative of each sample’s true behaviour.
When the entire suite of tests is considered, the Quartzite, Rhyolite and Quartz Diabase intrusions had the highest peak strength. When only the matrix failure is reviewed, the metacrystic gabbro far outperformed the other lithologies, although there appears to be a lack of testing results to substantiate this, given some lithologies did not fail solely through the intact material.

3.4.2 Sample Collection and Testing
Samples from pilot hole GT0020VCa were collected from 900-2000 mbgs for the purpose of estimation of compressive and tensile strength for each lithology. No samples above this depth were evaluated as no appreciable breakout was observed above this point (discussed further in Chapter 4). A representative number of samples from each lithology were also selected based on the importance which the rock type had on the frequency of breakout. If breakout appeared more prominent in one unit compared to another, that unit was sampled at closer intervals for consideration during testing. The ultimate goal for selection and preparation in each lithology was to achieve the minimum standards set by ASTM while also being conscious of the recommendations made by the ISRM on the same matter, given the slight discrepancies between the two.

A naming convention for all samples was established, and will be used throughout this dissertation. It identifies that the sample is taken for the analysis of breakout, in a given lithology and depth with a given sample number, and appears as follows:

BO-LITHOLOGY-DEPTH-SAMPLE#

Ex. BO-RHY-1300.1-55

3.4.2.1 Sample Selection Protocol
Prior to the selection of samples, a hierarchy was established to follow during the core logging process. As previously mentioned, the number of samples for each lithology was ideally dictated only by its importance to the process of borehole breakout observed in the hole. After this, the following considerations were made in this relative order when selecting samples for testing:
1) Sample contains no discontinuities and limited healed structures

2) Sample is greater than 5.0 m from a known contact or stiffness contrast from quartz veins

3) Sample is close to observed borehole breakout

4) Sample is a minimum of 30 cm length for the creation of one UCS, and two BTS specimens.

Although the selection criteria contains some degree of subjectivity, it was established in an attempt to minimize uncertainty associated with the test results, while also presenting a confident distribution as a function of depth within the borehole.

To preserve the intact conditions of the core throughout the transportation process, each sample was individually wrapped in plastic to preserve the moisture content, then surrounded by air bubble packing to limit the damage incurred (Figure 3-13). This was in an attempt to adhere to the suggested procedure outlined by the ISRM (1999).

![Figure 3-13 – Example of sample preparation from BO-MTBS-1152.9-15 prior to transport showing, a) the selected sample and, b) the final wrapped sample.]

3.4.2.2 Sample Preparation

Over the course of the testing campaign, 41 BTS and 47 UCS tests were prepared and tested for the characterization of the most recent pilot hole at the Queen’s University Geomechanics testing lab within
the Department of Geological Sciences and Geological Engineering. Throughout the entire process the
guidelines set out by the ASTM on sample preparation for the use in uniaxial compression tests (ASTM,
2013b) and Brazilian tensile strength tests (ASTM, 2013a) were followed.

One exception to the standards related to the core size tested. As the pilot hole was drilled using
HQ (63.5 mm) core until 1180 mbgs, NQ (47.5 mm) core until 1850 m and NQ3 (45 mm) until the hole’s
completion at 2000 m, none fully adhered to the specified 50 mm diameter in the ASTM standards.

One lab mandated preparation criteria, which is not a part of either the ASTM or ISRM standards,
was that all samples must be within 5% of the chosen length-to-diameter ratio. This standard was applied
for both the BTS samples at a ratio of 0.5:1 and the UCS samples at a ratio of 2.5:1.

3.4.2.3 Sample Testing Procedure
Testing of all laboratory samples was completed using a 2.2 MN capacity, computer servo-controlled,
hydraulic MTS machine. For the purpose of sample documentation, a set of photos for each was taken
prior to, and following each test. This involved photographs of the prepared sample, the sample once
instrumentation was attached, once the sample was within the load cell and the ultimate failed sample
after the test was complete. At this time, if any features of interest could be seen in the failed sample then
these were documented in more detail for use in description of the various failure mechanisms.

The initial portion of the unconfined compressive strength testing involved several calibration
samples to establish an axial deformation test rate that would meet the length of test standards set by the
ASTM. Once representative samples from each lithology were tested, a loading rate of 0.1 mm/min was
selected. From each unit, eight samples were chosen to be strain gauged, to measure deformation in the
lateral and axial sense. This involved the placement of four, 20 mm gauges with two being diametrically
opposed and placed along the axial direction, while the other two were at 90° to the axial gauges,
measuring strain in the circumferential direction. From all samples, the acoustic emissions (AE) was also
logged using a Physical Acoustics Corporation pocket AE system attached to two nano-sensors. These
sensors have a peak frequency of 270 kHz and were coupled to the sample through a brass fitting and
secured using elastic bands. This method was chosen as it allows for the bands to break and the sensors to detach without damaging them following the ultimate yield of the sample. The general setup from each UCS test can be seen in Figure 3-14.

Figure 3-14 – Example UCS test setup from sample BO-RHY-962.1-9 showing a) the relative circumferential location of each strain gauge and height of AE sensors and, b) the diametrically opposed location of each of the AE sensors.

Similar to the unconfined compressive strength test, the indirect tensile strength tests were completed in axial deformation control, at a rate of 0.04 mm/min. As can be seen in Figure 3-15, a curved seating system was used to uniformly distribute the applied load around a portion of each sample’s circumference. Only NQ and NQ3 sized core disks were tested in this series, as the ASTM standards specified an ideal disk diameter of 50 mm. Although testing HQ size disks would give some idea of the scale effect on tensile strength, no suitable samples, void of existing structures were found. On five samples from each lithology, a 10 mm lateral strain gauge was placed on either side, to measure deformation along the ideal failure plane. To preserve the samples after each test, a strip of tape was
loosely placed around the circumference of the sample, to allow for further interpretation of the crack propagation mechanisms.

Figure 3-15 – Example BTS test setup from sample BO-RHY-1602.6-53, showing the curved loading platens and location of one of the lateral strain gauges.

3.4.3 Measurement of Crack Occurrence Thresholds

Although standards have been published for the measurement of material properties such as the unconfined compressive strength and Young’s Modulus, much of the work and methods of determining crack damage thresholds in brittle rock rely solely on currently accepted research practices. Some of the initial work on crack development was done by Hoek (1964), Brace, Paulding and Scholz (1966) and Martin (1997) using the detailed measurement of strain data throughout intact laboratory tests. More recently, this work was furthered by authors such as Eberhardt et al. (1998) and Diederichs et al. (2004) on the measurement of acoustic emissions (AE) from crack initiation, and interaction in a sample during loading. Both these, and the strain based determination of crack occurrence have now become widely accepted in lab testing procedures. It is these, along with more recent developments by Ghazvinian et al. (2011), which have been used during sample testing and are described in the following sections. Table 3-3 provides a summary of measurement techniques used for the various strength estimates, and the associated standards or references for each.
<table>
<thead>
<tr>
<th>Material Property</th>
<th>Measurement Technique</th>
<th>Symbol</th>
<th>Standards/references</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unconfined Compressive Strength</td>
<td>Peak achieved stress during intact testing</td>
<td>UCS</td>
<td>ISRM (1999), ASTM (2013c)</td>
</tr>
<tr>
<td>Crack Initiation-CI</td>
<td>Non-linearity of lateral strain-axial stress curve-</td>
<td>CI-LS</td>
<td>Brace et al. (1966), Bieniawski (1967)</td>
</tr>
<tr>
<td></td>
<td>Reversal point of crack volumetric strain</td>
<td>CI-CVS</td>
<td>(Martin, 1997)</td>
</tr>
<tr>
<td></td>
<td>Inverse Tangent Lateral Stiffness</td>
<td>CI-ITLS</td>
<td>Ghazvinian et al. (2011)</td>
</tr>
<tr>
<td></td>
<td>Instantaneous Poisson's ratio</td>
<td>CI-IPR</td>
<td>Diederichs &amp; Martin (2010)</td>
</tr>
<tr>
<td></td>
<td>Uptake in AE</td>
<td>CI-AE</td>
<td>Diederichs et al. (2004)</td>
</tr>
<tr>
<td>Crack Damage-CD</td>
<td>Reversal in volumetric strain</td>
<td>CD-RVS</td>
<td>Martin (1997)</td>
</tr>
<tr>
<td></td>
<td>Non-linearity of Axial strain-axial stress curve-</td>
<td>CD-DAS</td>
<td>Brace et al. (1966), Bieniawski (1967)</td>
</tr>
<tr>
<td></td>
<td>Instantaneous Young's Modulus</td>
<td>CD-IYM</td>
<td>Ghazvinian et al. (2011)</td>
</tr>
<tr>
<td>Tensile Strength</td>
<td>Brazilian Tensile Strength</td>
<td>BTS</td>
<td>Bieniawski &amp; Hawkes (1978), ASTM (2013a)</td>
</tr>
<tr>
<td></td>
<td>Tensile strength from strain measurement</td>
<td>TS-SM</td>
<td>Perras (2014)</td>
</tr>
</tbody>
</table>

3.4.3.1 Crack Initiation Threshold (CI)

3.4.3.1.1 Lateral Strain/Axial Stress non-linearity- CI-LS

Through the use of strain measurement during the test cycle, the CI threshold was first shown by Bieniawski (1967) and Brace et al. (1966), to match the point of non-linearity of the lateral strain-axial stress curve (Figure 3-16-left). Although this was the first method for measurement, it has been shown
that the determination of this is somewhat subjective as the curve hardly ever reaches true linearity (Eberhardt, Stead and Stimpson, 1998; Ghazvinian, Diederichs and Archibald, 2011).

![Figure 3-16 – Evolution of the stress/strain as they correlate to the crack damage thresholds(after Diederichs et al. 2004).](image)

### 3.4.3.1.2 Crack Volumetric Strain - CI-CVS

The suggested use of crack volumetric strain for the identification of crack initiation was first made by Martin (1997). This is defined by the following:

\[
\varepsilon_{CV} = \varepsilon_{Vol} - \varepsilon_{EV} \tag{3-3}
\]

where \(\varepsilon_{CV}\) is the crack volumetric strain, \(\varepsilon_{Vol}\) is volumetric strain and \(\varepsilon_{EV}\) is the elastic volumetric strain.

The volumetric strain is a function of both the lateral strain (\(\varepsilon_{lateral}\)) and axial strain (\(\varepsilon_{axial}\)) as shown:

\[
\varepsilon_{Vol} = \varepsilon_{axial} + 2\varepsilon_{lateral} \tag{3-4}
\]

The elastic volumetric strain is then:

\[
\varepsilon_{EV} = \frac{1 - 2\nu}{E}(\sigma_1 - \sigma_3) \tag{3-5}
\]

where \(E\) is the Young’s Modulus and \(\nu\) is the Poisson’s ratio.
The theory behind the use of crack volumetric strain is that, during the compaction of a sample, at the first stages, the loading causes all pre-existing fractures to close, at which point the $\varepsilon_{CV}$ is stable. This means no dilation or compaction is occurring. As axial stress increases beyond this point, dilation of the specimen begins to occur, thus denoting the onset of cracking (Martin, 1997). Often times CI is chosen at the point of reversal in crack volumetric strain. Ghazvinian et al. (2012) however showed that this in practice may be quite difficult as the curvature at the maximum range is quite shallow. This is seen in Figure 3-17, as the top 1% of volumetric strain covers a testing range of 43 MPa for a test of Forsmark granite. For the purpose of testing, it has been determined that the reversal point of this 1% range should be chosen as the initiation of CI.

![Figure 3-17](image)

**Figure 3-17** – The measurement of CI-CVS from a test on Forsmark Granite, showing a) the selection of the maximum point along the crack volumetric strain curve and, b) the associated 43 MPa range associated with this technique when choosing the maximum 1% range (Ghazvinian, 2015).

### 3.4.3.1.3 Inverse Tangent Lateral Stiffness- CI-ITLS

The use of the inverse tangent lateral stiffness was proposed by Ghazvinian et al. (2011), which is a technique of clarifying the change of slope from the lateral strain-axial stress curve. This is done through
the use of a moving point regression technique to decrease the subjectivity of choosing the onset of non-linearity from the CI-LS method. The generalized form of this is as follows:

\[ \varepsilon_i \Delta = \frac{\Delta \varepsilon_i}{\Delta \sigma_{axial}} \]  

(3-6)

where \( \varepsilon_i \Delta \) is the inverse tangent lateral stiffness, \( \sigma_{axial} \) and \( \Delta \varepsilon_i \) is:

\[ \Delta \varepsilon_i = \varepsilon_{i+10} - \varepsilon_{i-10} \ (i=1,2,3...) \]  

(3-7)

\[ \Delta \sigma_{axial} = \sigma_{axial \ i+10} - \sigma_{axial \ i-10} \ (i=1,2,3...) \]  

(3-8)

where the interval, \( i \), is the number of points within the sliding window to calculate the ITLS and \( \sigma_{axial} \) is the axial stress. The chosen size of the window is subject to the user’s discretion, and should be adjusted to minimize noise. This method has been recommended to be used when a strong degree of precision can be obtained for the lateral strain and as such methods such as using extensometers or LVDTs are not suggested. As can be seen in Figure 3-18, once significant noise has been reduced, the crack initiation point is chosen as the first upwards inflection in the ITLS.

Figure 3-18 – CI-ITLS measurement technique from the use of lateral strain (Ghazvinian, 2015).
3.4.3.1.4 Instantaneous Poisson’s Ratio- CI-IPR

It was first suggested by Diederichs & Martin (2010) that the instantaneous Poisson’s ratio should systematically increase upon the initiation of cracking. This calculation is similar to the technique presented for the ISTL, where a query window is established over which the IPR is calculated. This is adjusted by the user to reduce the noise in the plot. The point of the first upwards inflection of instantaneous Poisson’s ratio is the CI threshold.

3.4.3.1.5 Uptake in AE- CI-AE

During the progression of brittle failure, the occurrence of extensional cracking provides a source of acoustic emissions (AE). It was shown by authors such as Scholz (1968) and Lockner (1993), that the change in increasing rate of AE occurrences denotes the onset of crack accumulation. Eberhardt et al. (1998) and Diederichs et al. (2004) have shown that when AE events suddenly increase above a background level, this denotes the crack initiation threshold. The formal method involves taking the cumulative number of cracks plotted against axial stress on a log scale to identify the point of sudden increase in AE activity. This is shown in Figure 3-19 as a range defined by the first point where AE events suddenly increase and where this increase becomes persistent (Diederichs, Kaiser and Eberhardt, 2004). This figure also shows the point of crack damage, which is discussed further in Section 3.4.3.2.4.

Figure 3-19 – Approach of using acoustic emissions to identify crack threshold limits, from the cumulative acoustic events versus axial stress on a log-log scale (Ghazvinian, 2015).
3.4.3.2 Crack Damage Threshold (CD)

3.4.3.2.1 Axial strain/axial stress non-linearity- CD-DAS
From the same work used to determine the CI-LS came the first determination of the crack damage threshold (W. F. Brace, Paulding and Scholz, 1966; Bieniawski, 1967). In a similar fashion to the CI method, this involves identifying the point of first non-linearity in the axial strain/axial stress plot, as shown in Figure 3-16. Similar to the estimation of CI-LS, this method has some uncertainty as a result of the fact that the line is never truly linear and therefore it is difficult to identify the first point of deviation from this supposed trend, thus producing a range for its estimate.

3.4.3.2.2 Reversal of Volumetric Strain- CD-RVS
In recognition of the limitations of CD-DAS, Martin (1997) proposed using the reverse in volumetric strain with respect to axial stress. This point of reversal represents the point when the rate of change in lateral strain begins to exceed that of the axial strain. It is intuitively satisfying that this is the point of crack coalescence as it is at this point when maximum sample dilation is also seen, leading to the increased lateral strain. The point of volumetric strain reversal can be seen in Figure 3-16.

3.4.3.2.3 Instantaneous Young’s Modulus- CD-IYM
The instantaneous Young’s Modulus approach (also known as instantaneous tangent modulus) is similar to the techniques described in the CI-ITLS and CI-IPR approaches where a window of data points is set to record the Young’s Modulus at a given point. This was first proposed by Eberhardt et al. (1998) and further expanded by Ghazvinian (2010), in recognition that the other methods of CD determination are subject to error at high levels of confinement. This operates under the principle that as trans-granular cracks begin to interact the system inherently becomes more “soft”, thus resulting in a notable decrease in Young’s Modulus. It is this point that indicates the onset of crack coalescence, which has been shown to be repeatable at various levels of confinement with respect to AE results (Diederichs, Kaiser and Eberhardt, 2004).
3.4.3.2.4 Second Uptake in AE- CD-AE

The second observed uptake in cumulative acoustic emission events during a sample’s compression has been shown to correlate well with other previously mentioned methods of CD threshold determination (Diederichs, Kaiser and Eberhardt, 2004). As can be seen in Figure 3-19, this increase is far more noticeable than the one associated with the onset of crack formation as it represents the point where the sample begins to fail towards its ultimate peak strength.

3.4.3.3 Tensile Strength

Given the difficulty of preparing and testing direct tensile strength samples, in practical engineering applications, Brazilian tensile strength tests are often done as an approximation of this important material property. From the review of existing data comparing direct tensile and Brazilian tensile strengths, Perras (2014) found that a factor $f$, could be used to relate the two, which is dependent on the rock type. This takes the following form:
\[ DTS = f \cdot BTS \]  \hspace{1cm} (3-9)

where the factor \( f \), is approximately 0.7 for sedimentary, 0.8 for igneous and 0.9 for metamorphic rocks.

From the further testing by Perras (2014), it was shown that this direct tensile strength coincides with the point of crack initiation during Brazilian tensile tests. If lateral strain gauges or AE sensors are placed on the sample during the test, this phenomenon could be recorded and thus used to estimate the true tensile strength during a Brazilian test. This can be done using the CI-LS method described in Section 3.4.3.1.1. This is intuitively satisfying as the mechanism of brittle failure is through the local tensional cracking leading to ultimate failure. It is this onset of extensional cracking which represents the true tensile strength of the material.

3.4.4 Testing Results

The following sections outline the results from the lab testing completed on pilot hole GT0020VCa for the purpose of identifying important material properties, for use in subsequent chapter’s numerical modelling. Although all Brazilian tensile strength tests were interpreted to have failed fully through intact material, the results from the UCS tests have been divided into three bins of; fully intact, partially intact and structural failure. These have been loosely defined as:

- Intact Failure (I)- Failure initiating and propagating in the axial direction without observable interference by through-going healed structures.
- Partially Intact Failure (PI)- A mixed mode of failure where fractures propagate in the axial direction until intersected by a feature such as healed structure or foliation, often at a low angle to the core axis.
- Structural Failure (S)- Failure initiates along a structure with no apparent failure through the intact material. This is often the case in samples that were seen to fail at low strengths.

Figure 3-21 shows a representative example of each failure type with a complete summary of results in Table 3-4. This summary only includes results acquired as part of this thesis, listing an average from all testing results and a review of intact and partially intact failure, as they are the focus of subsequent work.
In the case of determination of the various crack occurrence thresholds, an average between all methods was taken to produce the final estimates. This was only done with measurement techniques that yielded a conclusive result for each test. The tensile strength ($TS$) is a function of the measured Brazilian tensile strength as discussed in Section 3.4.3.3, with specific results shown in Section 3.4.4.2.

For a summary of testing results, Appendix B should be consulted, which provides all measured material properties.

Figure 3-21 – Examples of failure types observed during UCS testing showing a) fully intact failure (BO-RHY-972.2-11), b) partially intact failure (BO-MTBS-1151.0-14) and, c) structural failure (BO-QDIA-2000.1-74). Failure along existing structure is highlighted in red with, axial-dominated intact failure shown in blue.
Table 3-4 – Summary of average tensile and unconfined compressive strength testing results completed on pilot hole GT0020VCa for each major lithology. Tensile strength is considering the CI threshold from testing.

<table>
<thead>
<tr>
<th>Lithology</th>
<th>UCS#</th>
<th>BTS#</th>
<th>Density (g/cm³)</th>
<th>Young’s Modulus (GPa)</th>
<th>Poisson’s Ratio</th>
<th>UCS (MPa)</th>
<th>CI (MPa)</th>
<th>CI/UCS</th>
<th>CD (MPa)</th>
<th>CD/UCS</th>
<th>TS (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Average Results</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MTBS</td>
<td>17</td>
<td>10</td>
<td>3.01</td>
<td>96.2</td>
<td>0.26</td>
<td>198.0</td>
<td>79.6</td>
<td>40%</td>
<td>146.3</td>
<td>74%</td>
<td>14.3</td>
</tr>
<tr>
<td>MTGB/MXGB</td>
<td>11</td>
<td>10</td>
<td>3.03</td>
<td>97.7</td>
<td>0.27</td>
<td>213.9</td>
<td>92.6</td>
<td>43%</td>
<td>174.9</td>
<td>82%</td>
<td>14.5</td>
</tr>
<tr>
<td>RHY</td>
<td>13</td>
<td>11</td>
<td>2.73</td>
<td>71.9</td>
<td>0.22</td>
<td>196.2</td>
<td>89.1</td>
<td>45%</td>
<td>170.0</td>
<td>87%</td>
<td>12.7</td>
</tr>
<tr>
<td>QDIA</td>
<td>6</td>
<td>10</td>
<td>2.93</td>
<td>82.5</td>
<td>0.25</td>
<td>251.0</td>
<td>117.4</td>
<td>47%</td>
<td>213.5</td>
<td>85%</td>
<td>14.6</td>
</tr>
<tr>
<td><strong>Intact only</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MTBS</td>
<td>5</td>
<td>10</td>
<td>2.96</td>
<td>97.6</td>
<td>0.26</td>
<td>232.3</td>
<td>95.9</td>
<td>41%</td>
<td>179.9</td>
<td>77%</td>
<td>14.3</td>
</tr>
<tr>
<td>MTGB/MXGB</td>
<td>2</td>
<td>10</td>
<td>3.03</td>
<td>91.4</td>
<td>0.29</td>
<td>329.5</td>
<td>139.0</td>
<td>42%</td>
<td>242.5</td>
<td>74%</td>
<td>14.5</td>
</tr>
<tr>
<td>RHY</td>
<td>6</td>
<td>11</td>
<td>2.73</td>
<td>80.4</td>
<td>0.24</td>
<td>210.8</td>
<td>88.3</td>
<td>42%</td>
<td>160.4</td>
<td>76%</td>
<td>12.7</td>
</tr>
<tr>
<td>QDIA</td>
<td>3</td>
<td>10</td>
<td>2.94</td>
<td>81.6</td>
<td>0.27</td>
<td>297.1</td>
<td>127.4</td>
<td>43%</td>
<td>240.4</td>
<td>81%</td>
<td>14.6</td>
</tr>
<tr>
<td><strong>Partially intact</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MTBS</td>
<td>8</td>
<td>-</td>
<td>3.05</td>
<td>95.1</td>
<td>0.27</td>
<td>177.4</td>
<td>72.0</td>
<td>41%</td>
<td>117.0</td>
<td>66%</td>
<td>-</td>
</tr>
<tr>
<td>MTGB/MXGB</td>
<td>5</td>
<td>-</td>
<td>3.03</td>
<td>93.6</td>
<td>0.25</td>
<td>180.4</td>
<td>85.1</td>
<td>47%</td>
<td>167.9</td>
<td>93%</td>
<td>-</td>
</tr>
<tr>
<td>RHY</td>
<td>4</td>
<td>-</td>
<td>2.72</td>
<td>71.7</td>
<td>0.22</td>
<td>200.9</td>
<td>85.2</td>
<td>42%</td>
<td>168.9</td>
<td>84%</td>
<td>-</td>
</tr>
<tr>
<td>QDIA</td>
<td>2</td>
<td>-</td>
<td>2.92</td>
<td>81.6</td>
<td>-</td>
<td>197.9</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

3.4.4.1 Unconfined Compressive Strength

As previously described, during the completion of UCS testing, not all samples were subject to monitoring with strain gauges. In all cases, acoustic emissions were monitored for use in estimating important crack thresholds. These samples formed the basis for estimating CI and CD and their relative proportions to the peak strength of each sample. In the case of CI this varied from 40%-50% of the UCS while CD was in the range of 70%-80% when considering only intact failures. These ratios fall well within the values reported in literature for brittle rock by authors such as Brace et al. (1966), Bieniawski (1967), Lockner (1993), Martin (1993) and Holcomb & Costin (1986).

Some caution was required when estimating the Young’s Modulus from samples which were not strain gauged. Often in practice, when no strain measurements are available, the axial strain can be measured from the displacement of the LVDT. When this was done in the case of the testing, this resulted in stiffness estimates of less than half that of the samples that had been strain gauged. This is most likely
due to a softening effect created by the LVDT system, which has multiple sets of hydraulic parts applying the load across various material types, which could ultimately lead to this apparent behaviour. Other researchers confirmed this same finding from their tests (personal communication- Jaczkowski, Ghazvinian). As such, stiffness was only estimated on samples which were instrumented.

Figure 3-22 shows the relative distribution of peak strength for all tests and failure types. Each type of failure has also been highlighted in the backdrop of the plot to show the relative distribution as strength increases. It should come as no surprise that samples exhibiting lower strength were the ones that tended to fail as a result of structure, while the more competent samples above 300 MPa almost exclusively failed through the intact material. The average $\text{UCS}$ for each rock unit varied from 196.2 MPa (RHY) to 251.0 MPa (QDIA) when all samples are considered, but reached a maximum average of 329.5 MPa (MTGB) when only the intact samples were used.

**Figure 3-22 – Distribution of unconfined compressive strength as a function of lithology, showing the normal distribution from each dataset. Backdrop of plot shows relative proportion of samples that failed along structure, partially intact or fully intact.**
The distribution of failure type was further refined to consider which rock type was more prone to failure along healed structures. From Figure 3-23 it is seen that both the rhyolite and quartz diabase intrusions experienced the greatest proportion of intact failure, while both the metabasalt and metagabbro showed an increase in structural or partially intact failure. The result from the RHY comes as a bit of a surprise, given the foliated nature of the rock. This could be explained by the fact that the intensity of foliation increases in the rhyolite as a function of depth, with most samples from this lithology coming from shallower levels in the pilot hole. The relatively low amount of intact failure in the metagabro/metacrystic gabbro was also expected given the amount of healed structure found in these intervals, as previously described in Section 3.2.1.3 and shown in Figure 3-4.

![Figure 3-23 – Distribution of failure type based on lithology, from laboratory unconfined compressive strength in borehole GT0020VCa.](image)

When comparing the crack thresholds during intact and partially intact failure, it becomes immediately apparent that the $CD/UCS$ ratio measured during partially intact failure is measurably higher than that occurring during solely intact sample failure. This is shown in Figure 3-24, where this ratio exceeds 90% for the MTGB rock type. Despite this, a similar trend is not observed in the $CI/UCS$ ratio, which remains relatively constant between failure types. This could be indicative of the fact that crack
initiation in the samples occurs at a relatively uniform stress and is therefore not uninfluenced by the presence of structure in the rock. The magnitude of each crack damage threshold, is the same or greater for the partially intact failure compared to the fully intact failure samples. When considering both this and the difference in ratios, this result could be attributed to the fact that the stress where cracks begin to interact and coalesce in each rock type is the same, although this phenomenon acts to quickly exploit existing structure in the sample, thus causing ultimate failure at much lower strengths in samples having existing healed structure. In some cases, samples failed along structure prior to reaching $CD$. This was shown by the lack of deviation in strain data or increase in AE activity during the test to support that the $CD$ threshold had been reached. This further demonstrates some of the issues of solely relying on $UCS$ as a measure of strength, as it is scale and load-rate dependent and shouldn’t be considered the actual in situ strength of a brittle material surrounding an excavation (Diederichs, 2007). This is where the determination of $CI$ and $CD$ is imperative when testing brittle materials, for use in modelling of excavation scale phenomenon.

When comparing the various core sizes which were tested (HQ/NQ/NQ3), there was a considerable drop of strength recorded in the HQ samples. This can be attributed to the scale effect by a greater volume of heterogeneous features in the larger samples. This allows cracks to exploit these areas, thus allowing for ultimate yield at lower levels of stress. This was particularly evident in HQ samples which had a greater amount of healed structure, which became the catalyst for partially intact failure in most cases. It was also noted during sample collection that the interval from 950-1075 m contained a greater amount of healed structures within the intact rock. Given that the HQ sized core occurred only until 1178 m, selection of these sizes of sample could largely only occur within this interval. In the case of the smaller diameter core (NQ/NQ3) there was a greater length of borehole from which to select the most intact samples for testing. This most likely had an impact on the perceived “scale effect” on rock strength.
3.4.4.2 Brazilian Tensile Strength

During the Brazilian tensile strength testing campaign, five samples from each lithology were instrumented with two lateral strain gauges placed along the mid-point of each face of the specimen. This was done to capture the occurrence of crack initiation during loading, for use in estimating true tensile strength. As described by Perras (2014), this is done in recognition that cracking in a brittle material is evidence that the tensile strength at the grain scale is being exceeded, thus leading to extensional cracking. Beyond this point in a Brazilian test, any further loading is merely failing the sample through compression. This phenomenon is shown in Figure 3-25 where two samples show the characteristic axial splitting in addition to partial oblique fractures, which originate at the specimen’s circumference and terminate upon intersection with the axial fracture. It is this low angle crack, that was seen in most samples, which occurs at peak strength and is further developed as a result of compression along the curved loading surface, which is in contact with the sample.

Figure 3-24 – Summary of strength damage thresholds as a function of lithology, with crack occurrence thresholds being plotted as ratio to the peak strength (line) for intact failure (left) and partially intact failure (right).
Figure 3-25 – Example showing Brazilian tensile strength samples a) BO-MTGB-1335.4-29b and b) BO-RHY-1482.0-45a, with axial fracture and subsequent oblique fractures at an angle to the through-going failure.

From the strain gauge data, the approach of evaluating non-linearity of the lateral strain with respect to axial stress was used to select the crack initiation point. This is shown in Figure 3-26, where the pair of strain gauges produces an average CI of 14.1 MPa while the test had a peak stress of 16.4 MPa, thus producing an estimated tensile strength of 84% of the Brazilian tensile strength. This result fits within the ranges for igneous to metamorphic rocks as presented by Perras (2014). The results from all estimates of DTS/BTS ratios are summarized in Table 3-5, where the ratio varied from 72% (QDIA) to 80% (MTBS) of the peak test strength. The coefficient of variation from each rock type never exceeded 6% for the estimation of the strength ratio.
Figure 3-26 – Demonstration of the determination of crack initiation during a Brazilian tensile strength test for sample BO-MTBS-1181.4-19a, for the determination of tensile strength.

Table 3-5 – Observed ratio of tensile strength (as indicated by crack occurrence) to the peak tensile strength during Brazilian tensile strength testing for each lithology. For samples without strain gauges, these ratios were applied for a final estimate of tensile strength, as reported in Table 3-4.

<table>
<thead>
<tr>
<th>Litho.</th>
<th>DTS/BTS Avg.</th>
<th>DTS/BTS Std.dev</th>
<th>DTS/BTS COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>MTBS</td>
<td>0.80</td>
<td>0.05</td>
<td>6%</td>
</tr>
<tr>
<td>MTGB</td>
<td>0.78</td>
<td>0.03</td>
<td>4%</td>
</tr>
<tr>
<td>RHY</td>
<td>0.77</td>
<td>0.04</td>
<td>6%</td>
</tr>
<tr>
<td>QDIA</td>
<td>0.72</td>
<td>0.03</td>
<td>5%</td>
</tr>
</tbody>
</table>

For the final estimation of tensile strength presented in Table 3-4, the ratios of tensile strength to Brazilian tensile strength for each rock type was applied to all samples which weren’t strain gauged. This created the distribution of strengths shown in Figure 3-27 where the bounds on tensile strength were 3.9 MPa and 20.1 MPa, both in the MTGB. This range of tensile strength estimates in the Metagabbro could...
be attributed to the inherent compositional variability of this rock type. As previously described, the MTGB had two end members in grain size, from the coarser grained, poikilitic Metacrystic Gabbro to a fine grained Metagabbro. These weren’t differentiated as distinct lithologies as the grain size changes appeared to often be gradational in nature with no distinct transition. For the lower bound strength, this occurred through a sample with healed structure (BO-MTGB-1945.6-65a), causing failure at a lower stress than what was seen during testing of more intact samples. Despite this observed range of tensile strength in the MTGB, it did however show the lowest variability in estimated CI from the samples that were strain gauged (Table 3-5).

A complete summary from each test with accompanying estimates of crack initiation, where applicable, can be found in Appendix A.

![Distribution of tensile strength per lithology, showing normal distribution of each data set.](image)

**Figure 3-27 – Distribution of tensile strength per lithology, showing normal distribution of each data set.**

### 3.4.5 Discussion of Test Results

#### 3.4.5.1 Observations of Failure Mechanism

Throughout the testing of both compressive and tensile strengths, one of the most evident observed characteristics was the suddenness and intensity at which the samples tended to fail. This was particularly
obvious in the larger HQ samples, where the applied loads exceeded 1000 kN in some of the more competent specimens. Figure 3-28 demonstrates two representative samples, where both exhibit such sudden loss in strength following peak values, that the logger was unable to capture any post peak behaviour. In the case of sample BO-RHY-962.1-9 (orange), the drop-in stress was rapid enough, such that the data recorder wasn’t able to capture any apparent drop in stress.

![Figure 3-28](image)

**Figure 3-28 – Comparison between two samples of rhyolite showing inherently brittle behaviour.**

The orange line is taken from an HQ sample, while the blue is from a NQ sample.

These tests also often failed with such energy that no portions of the sample remained on the load cell platen. This made documentation of the failure sometimes difficult, although it also resulted in some of the observed phenomenon shown in Figure 3-29. In the case shown in Figure 3-29a, a sample of Quartz Diabase dyke effectively spalled into small planar fragments. Once this sample failed axially, half of it remained on the load cell and progressively formed this “onion-skin” like failure, until total yield. Figure 3-29b is from the same sample which is documented as the orange line in Figure 3-28, where the failure was catastrophic enough to eject all portions of the sample from the platen seat, excluding the sliver-like fragment shown in the image. Figure 3-29c demonstrates similar characteristics to the QDIA sample,
where the sample initially failed in the axial direction, followed by the spalling of small wedges, which can be observed along the middle of the sample. In both the case of a) and c), these fragments were kept together due to the elastic bands that secured the AE sensors to the sample.

Figure 3-29 – Examples of observed intact failure from samples a) BO-QDIA-1640.2-56, b) BO-RHY-962.1-9 and, c) BO-RHY-1479.8-44.

3.4.5.2 Influence of Foliation on Failure

From the description of core, the rhyolite has a preferentially aligned foliation, with some of the UCS tests failing along these planes of weakness. This is particularly true in the intervals of RHY below 1500 m, where a distinct compositional change is seen, leading to a more prominent foliation. This is observed as an increase in a darker and more friable mineral (likely an amphibole) as lenses within the sample, forming the foliation. Upon testing of the RHY in this same lithological horizon, the observed mechanics of failure further show the influence of the foliation. As can be seen in Figure 3-30, these
samples show a dominant yielding along foliation and lack of axial failure, which is prominent in most of the other tests. During testing, three samples of rhyolite demonstrated this behaviour, which resulted in strengths 50% lower than other samples in less-intensely foliated rhyolite. The effects of this on breakout occurrence and further prediction of stress is discussed in detail throughout the upcoming chapters.

![Image of sample failure](image)

**Figure 3-30 – Observed failure of sample BO-RHY-1610.8-54 along preferentially aligned foliation.**

### 3.4.5.3 Depth Dependent Strength Variation

Strength as a function of depth was also reviewed (Figure 3-32). As denoted by the hashed blue line, the HQ core that was tested as part of the *UCS* campaign showed a slightly lower strength. At the point where core size was reduced to NQ (1180 m), there is a sharp jump by approximately 40 MPa in peak strength, while *CI* and *CD* remained relatively unaffected. At 1400-1550 m, both *UCS* and *CD* show an increase in strength, with the greatest corresponding to an interval of rhyolite between 1475 and 1520 m. Between 1500-1600 m the crack damage and peak strength profiles converge on each other. This may be an indication that, despite the most rigorous sample selection process, there may have been some incipient stress-induced core damage. This damage would occur at the grain scale and could act to reduce the peak
test strength to the $CD$ threshold (Martin, 1997; Diederichs, Kaiser and Eberhardt, 2004). An example of grain-scale damage within this interval is seen in Figure 3-31. In this case a quartz inclusion within a sample of rhyolite shows stress-relief damage, which is a precursor to core disking.

Figure 3-31 – Sample of rhyolite at 1585 m depth in pilot hole GT0020VCa showing grain-scale stress induced damage within the quartz inclusion.

For the tensile strength results, no samples were prepared in HQ sized core. This was done to conform with the testing standards, which ideally required a 50 mm diameter sample. From the profile of tensile strength with depth, tensile strength shows relatively little correlation with the compressive strength. The only place where both the tensile strength and peak strength show some conformity is at 1600 m, where there is a decrease in both, corresponding to the highly-foliated interval of rhyolite.
3.4.5.4 Pilot Hole Comparison

The testing results outlined in Section 3.4.1 for pilot hole FNX-1204 and those completed as part of this dissertation show a remarkable contrast in values (Table 3-6). Although the samples are from different pilot holes, their relative location differs by less than 100 m with no major faulting or variation in lithologies between them.

It is particularly troubling that the average Poisson’s ratio was 55% larger, the Young’s Modulus 49% larger, and UCS 40% larger in pilot hole GT0020VCa as compared to the results from FNX-1204. Although special variability in rock strength and stiffness could explain some discrepancy between the testing results, it is difficult to explain this great of a difference on solely that. This is most obvious when looking at the Poisson’s Ratio, where the obtained average between them was respectively 0.11 and 0.25 between FNX-1204 and GT0020VCa. In the former of these two, this corresponds to the lower bound
Poisson’s ratio of sandy gravel (Das, 2002) or conglomerate (Gercek, 2007), while results from the testing of GT0020VCa closely fits the reported average Poisson’s ratio of most crystalline rock (Gercek, 2007).

Table 3-6 – Comparison of lab testing results between pilot holes GT0020VCa and FNX-1204, considering all tested samples. Red shades denote a greater difference between holes with the mean difference of each material property shown at the bottom. Refer to Table 3-2, Table 3-4 and Appendix A for full summary.

<table>
<thead>
<tr>
<th>Lithology</th>
<th>GT0020VCa</th>
<th>FNX-1204</th>
<th>% Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Young's Modulus</td>
<td>Poisson's Ratio</td>
<td>UCS</td>
</tr>
<tr>
<td></td>
<td>Gpa</td>
<td>MPa</td>
<td>Gpa</td>
</tr>
<tr>
<td>MTBS</td>
<td>96.2</td>
<td>0.26</td>
<td>198.0</td>
</tr>
<tr>
<td>MTGB/MXGB</td>
<td>97.7</td>
<td>0.27</td>
<td>213.9</td>
</tr>
<tr>
<td>RHY</td>
<td>71.0</td>
<td>0.22</td>
<td>196.2</td>
</tr>
<tr>
<td>QDia</td>
<td>82.5</td>
<td>0.25</td>
<td>251.0</td>
</tr>
<tr>
<td>Mean</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Without being a part of the sample selection, preparation and testing for FNX-1204, it is difficult to come to an exact conclusion as to why these differences may exist. Given that the peak strength from pilot hole FNX-1204 more closely relates to the crack damage threshold of the more recent test samples, this shows that the core could have some incipient stress-induced damage or have been damaged during transportation. The effects of this were briefly discussed when reviewing the strength as a function of depth profile from GT0020VCa, where a similar phenomenon was observed through a 50 m interval from 1550-1600 m. The cracks could also explain the lower Poisson’ ratio as the horizontal flaws would preferentially close during sample loading, leading to an increased axial strain in the sample.

Due to the limited knowledge of the conditions in the previous testing cycle, only the results which were obtained from testing of the most recent pilot hole will be used as inputs for material properties during the remainder of this thesis. This provides continuity to the analysis as everything from sample selection and preparation to testing and interpretation was done solely by the author. The impact which this may have on stress estimation between the two holes is discussed in further detail in Chapter 6.
Chapter 4

A Methodology for Evaluation of Borehole Breakout

4.1 Introduction

ATV data has been gathered along the length of both pilot holes to assess not only the frequency and orientation of fractures, but also the stress induced borehole breakout. For the purpose of this study, breakout characteristics were compiled for the interval of 875–2,000 mbgs as above this depth no appreciable stress-induced breakout was observed in either borehole. In addition to the ATV survey, a three-arm caliper was used to verify point deviations in borehole diameter and cross reference the observed deformations from the geophysical survey.

4.1.1 Data Acquisition

During each survey, measurement points were taken every 2.5° around the circumference of the hole and at every 2 mm depth to produce the continuous false image of the borehole wall. This was done using the multi-echo technique, which captures the wave response in the form of its amplitude and travel time changes from the initial emitted beam (Deltombe and Schepers, 2004). As is done in most ATVs surveys, post processing of the data was done to reduce noise that is non-indicative of the geological conditions. This includes removing the acoustic window reflection, which is the machine’s protective case that the beam must first exit prior to reflecting off the borehole wall. The signal echo within the borehole is also removed in cases where the emitted beam reflects off multiple surfaces in the borehole as a result of irregular geometries, which causes attenuated amplitude returns that can be discounted (Deltombe and Schepers, 2004).

Over the length of each pilot hole, artifacts in the data, which could not be corrected for, accounted for under 1% of each borehole’s entire length. Figure 4-1 shows such anomalies from pilot hole GT0020VCa, demonstrating borehole enlargements from points where the borehole was wedged to correct for drill hole wander (Figure 4-1a) and tool speed irregularities in the intact borehole (Figure 4-1b).
and through a fault zone (Figure 4-1c). In the case of the tool speed irregularities, this can be caused by the tool sticking to the side of the borehole, thus causing it to rotate at a slower or faster rate than the required amount. This causes an uneven distribution of measurement locations, often leading to wave like features in the data (Lofts and Bourke, 1999).

![Figure 4-1 – Examples of artifacts in the ATV survey of pilot hole GT0020VCa showing a) location of wedging of the borehole, b) tool speed irregularities leading to wavy features and c) tool speed irregularities, from the intersection with large open fault features.](image)

4.2 Breakout Selection and Interpretation

Prior to the selection of breakout profiles to be used in the analysis, each was first reviewed to meet a set of criteria. This was done to maximize the uniformity in rock conditions at the point of breakout, while reducing the effect of heterogeneity such as discontinuities and lithological contacts on the formation of breakout. This involved the review of over 750 profiles between the two pilot holes, of which less than 55% were selected to be used for full characterization.
Breakout was only considered if there was greater than a 2.5% increase in travel time relative to the mean borehole travel time. Deviations smaller than this were not recorded, as their scale approached that of the maximum rock grain size. Similar values for cut-off travel times were used by Walton et al. (2015), based on their experiences in comparable brittle rock. A summary of the selection criteria includes:

- Breakout that does not appear to be influenced by:
  - Major fault structures
  - Sub-vertical jointing
  - Contacts with quartz veins
- Has greater than a 2.5% increase in ATV travel time from borehole wall to breakout apex.
- Is not intersected along the length of breakout by a discontinuity.
- Has less than a 0.75°/cm deviation in breakout azimuth.

Of the profiles which met the criteria, each were characterized based on the average opening angle and breakout depth, normalized to the mean travel time of the undisturbed borehole (Figure 4-2). This process removes any uncertainty with converting from travel time to distance caused by the presence of drilling muds or water saturation in the borehole. Given that the borehole diameter in GT0020VCa also changed from HQ (63.5 mm) to NQ (47.6 mm) at 1,180 m, the approach of normalizing breakout depths allowed for each breakout to be assessed in parallel regardless of the borehole. This also allowed for two dimensionless descriptors of breakout, for ease when scaling the numerical model geometry (discussed further in Chapter 5).
Excel was used to semi-automate the process of converting the raw travel time data from the ATV survey to the normalized breakout depth. This first involved selecting in WellCad the depth interval that contained each breakout of interest. The raw travel times were then imported to Excel, where the average travel time to the undisturbed borehole wall was determined. This was done by taking travel time measurements along the entire circumference of the borehole at multiple places above and below each breakout. This was done to reduce the effects of elastic borehole deformation on the final measurement of undisturbed borehole travel time. Each value of travel time was then normalized to this value of undisturbed borehole travel time to produce a false colour image such as the one seen in Figure 4-3a. Conditional formatting was then used to provide colour to the image, at which time the dataset was re-imported into WellCad to create the final normalized breakout depth plot that can be seen on the right side of Figure 4-2.

To evaluate the actual extent of breakout (opening angle and normalized depth), a filter was passed to remove any points that remained undisturbed or showed a decrease in borehole diameter. This
produced the image in Figure 4-3b, where an automated process then counted the number of cells in each row that could be attributed to an increase in borehole diameter while also recording the maximum breakout depth from each row of interest. This was all done considering the threshold described above that breakout must exceed 2.5% to be accepted. Through visual inspection, the top and bottom 20% of each breakout profile were discarded with the remainder used to create an average estimate of breakout opening angle and normalized depth. In the case of Figure 4-3, this created an estimate of 55° opening angle and 1.45 depth of breakout.

![Figure 4-3](image)

**Figure 4-3 – Processing technique used to convert the raw travel time data from ATV survey to a) normalized borehole travel time, with further filter to remove any zones of contraction to show b) the geometry of breakout. Example is from pilot hole GT0020VCa at 1609.0 m depth, and was used to create the post processed image in Figure 4-2.**

The criteria of maximum breakout azimuth deviation was one which was qualitatively determined from the review of profiles that were notably disturbed by discontinuities or other heterogeneities. Once the 0.75°/cm was established, the location of maximum breakout depth from each row of data (Figure 4-3b), was recorded and plotted as a function of depth. An example of this, with the underlain image of breakout is shown in Figure 4-4, where the green trace represents the location of maximum breakout
depth along its length. This case produced an average deviation of 0.08⁰/cm, which represents one of the most regular profiles observed throughout either pilot hole.

Figure 4-4 – Example of determination of the maximum breakout depth trace, for use in assessing regularity of breakout and the influence of heterogeneity. Image taken from pilot hole GT0020VCa at 1609.0 m depth in RHY.

4.3 Pilot Hole Breakout Characteristics

Upon applying the criteria previously described, 255 and 212 breakouts were respectively chosen from FNX-1204 and GT0020VCa for the back analysis of stress. The breakout results from the ATV survey can be seen in Figure 4-5 and Figure 4-6, showing the global average azimuth of breakout being approximately north–south, corresponding to a maximum horizontal principal stress in the east–west direction. Although stress variability in the Sudbury Basin has been documented, this result is consistent with work done along the Southern Range (Trifu and Suorineni, 2009; Snelling, Godin and McKinnon, 2013; Walton et al., 2015) and regionally in the Canadian Shield (Herget, 1987; Diederichs, 1999).
As seen in Figure 4-5, there are minor rotations in stress observed at 1,000 m near the upper contact with the RHY and once again at 1,600 m within a tightly spaced sequence of lithologies. As shown by the distance weighted averages (solid lines), at 1,000 m the azimuth of breakout rotates up to 15° in an eastward sense, while at 1,600 m it shows a rotation in opposite directions between the two boreholes. Similar to the observed geometry changes, the rotation in breakout azimuth may suggest that the in-plane principal stress ratio is close to unity at these locations. This could allow small heterogeneities or anisotropy in the rock to play a greater role in influencing the geometry and orientation of breakout, particularly in the foliated rhyolite unit.

Along the profile of each borehole, the shape of breakout (opening angle and breakout depth) shows some degree of irregularity. As shown in Figure 4-6, the mean breakout angle is 32° with a breakout depth of 1.17 times the borehole radius. This relatively small-scale of breakout may further suggest that the average horizontal stress ratio is quite low, which supports the variability seen in
breakout geometry and azimuth. The areas showing large variation in geometry may also be a result of the influence of rock anisotropy or heterogeneity such as foliation or inclusions which could locally influence the shape of breakout. Unlike the breakout azimuth, which is ideally only controlled by the far-field stress orientation, the shape of breakout is entirely dictated by stress magnitudes and mechanical rock properties (assuming a homogeneous/isotropic medium).

Figure 4-6 – Opening angle (left) and normalized breakout depth (right) of measured breakouts from ATV survey. Measurement locations are shown as points with the mean breakout angle and breakout depth as the hashed line. An inverse square mean used for trend analysis (solid line).

4.4 Validation with Core-Disking Geometry

Despite some ongoing uncertainty regarding the prediction of stress magnitudes from core disking, more conclusive research has been done regarding how the disk geometry can be used to evaluate the orientation of principal stress in the cross-sectional plane of the borehole. Based on the height distribution around the periphery of the end surface of each disk, forming a saddle-shape, multiple authors were able to deduce the maximum and minimum in-plane stress directions (Dyke, 1989; Haimson and Lee, 1995; Li and Schmitt, 1998). In particular, Haimson and Lee (1995) showed through laboratory induced core
disking that the concave axis gives the direction of maximum principal stress, while the convex axis is the minimum principal in-plane stress (Figure 4-7). This was further confirmed by Li and Schmitt (1998), who, through finite element modelling, observed the fracture trajectory taken during core disking, which formed the same characteristic saddle-shape. Although these studies were limited to the case where one of the principal stresses are along the core axis, Matsuki et al. (1997) showed that this phenomenon could still occur when stresses are acting at some angle to the core axis, although deviation in the idealized surface geometry would occur.

![Figure 4-7 – The saddle-shaped core surface created through the disking phenomenon, with the geometry denoting the orientation of principle stresses (Matsuki et al., 2004).](image)

**4.4.1 Disking Characterization- GT0020VCa**

Below 1700 mbgs in pilot hole GT0020VCa, where core was oriented (subject to fault zone intersection), foliation orientation and the axis of concavity within disked intervals were measured. Along this interval, 24 occurrences (44 m) of disking were logged, primarily within large (>50cm) quartz veins. As depth increased, disking was more frequently seen in the foliated footwall rocks. Logging included the measurement of disk thickness in addition to the axis of symmetry of each disk, relative to the core orientation line. Figure 4-8 demonstrates two instances where the measurement of the axis of concavity is quite apparent, within an interval of MTBS and a quartz vein. This idealized geometry wasn’t always the case, as sometimes intersecting structure or other heterogeneities cause an irregular shape to the disks.
Figure 4-8 – Samples taken of disk geometry from MTBS at 1392.9 m (a & b) and a quartz vein at 1583.7 m (c & d). Images a) and c) show an oblique view with the concave axis (red-\(\sigma_1\)) and convex axis (purple- \(\sigma_3\)), while b) and d) show an in-line view, parallel to the concave axis.

Where possible, each disk’s shape was logged, and is summarized in Figure 4-9, where the concave axis is shown in red and the convex is in blue. Although some spread in data is seen, the observed mean orientation of \(\sigma_H\) in the horizontal axis is 88° (East-West) while the orientation of \(\sigma_h\) is 2° (North-South). From the azimuth of breakout seen along the same interval of the hole, it can be seen how closely each of these measurement techniques closely correlate. Another way of assessing the performance of the technique is by reviewing the measurement difference between the orientation of the concave and convex axis. Ideally, this would always be 90° and it can be seen in Figure 4-9, that this is almost the case with a mean difference of 88°. The variation seen in this measurement can be as a result of the influence of foliation, structure or other inclusions in the rock, leading to an irregular disk profile. It can also represent a source of measurement error, although in all likelihood the closeness of fit between
the ideal and observed values may be due to measurement bias during the core logging process. This arises from the fact that during measurement, the ideal result of difference between the two axes is known, which inherently influences the measurement of the second orientation with respect to the first.

Figure 4-9 – Measurement results of disk geometry from oriented core showing the concave ($\sigma_1$) and convex ($\sigma_3$) axis (left) and the relative difference in angle between the two orientations (right). The data in orange is the breakout azimuth measurements and moving average taken from ATV data as a comparison.

4.5 Illustrative Study of Rock Anisotropy

Of the five lithologies that have been examined for breakout, the rhyolite is one which demonstrates the most variability in breakout azimuth and opening angle. From the description of core, this rock type also has a preferentially aligned foliation, with some of the UCS tests failing along these planes of weakness. This is particularly true in the intervals of RHY below 1500 m, where a distinct compositional change is
seen, leading to a more prominent foliation. This is seen as an increase in a darker and friable mineral (likely an amphibole) as lenses within the sample that form the foliation (Figure 4-10b). The first indication that foliation may have an influence on breakout occurrence was through the review of the ATV data from pilot hole GT0020VCa. This can be seen in Figure 4-10a where breakout isn’t fully formed, but is preferentially aligned at some angle to the borehole axis. Upon testing of the RHY in this same lithological horizon, the observed mechanics of failure further show the influence of the foliation. When this is compared with some of the more intact samples outlined in Chapter 3, this unit shows a lack of axial failure, which was prominent in most of the other tests.

**Figure 4-10** – Illustration of the effects of foliation on a) breakout occurrence in pilot hole GT0020VCa at 1685.0 m depth and b) the mechanics of UCS test failure on the same sample.

### 4.5.1 Discussion of Foliation Influence

Much work has been done in sub-horizontally bedded shale to show the effects that systematic planes of weakness have on the occurrence of breakout (Zhang, 2013; Meier *et al.*, 2015). These authors have
shown that the orientation of breakout will tend to rotate towards being aligned with the dip direction of
the bedding, despite the orientation of the major principal stress. This trend becomes more apparent as the
inclination of the planes of weakness increases. Despite the inherent mechanical differences between
shale and the rock types in question for this study, the presence of micro-structure at high angles to the
borehole should have similar effects on breakout geometry.

Modelling the effects of foliation was done using the finite-element software RS2 -9.0. To
replicate brittle failure within the continuum, the damage initiation and spalling limit approach developed
by Diederichs (2007) was used. This considers the peak strength, tensile strength and \( CI \) to produce a
composite strength envelope that is used in RS2. The material property inputs from lab testing and the
resulting outputs used in modelling are summarized in Table 4-1, including all other pertinent values. In
this demonstration of the influence of foliation, the strength of rhyolite from testing was used.

Table 4-1 – Summary of material properties and model inputs using the DISL approach.

<table>
<thead>
<tr>
<th>Material Properties</th>
<th>Model Input</th>
</tr>
</thead>
<tbody>
<tr>
<td>( E ) (GPa)</td>
<td>( c_i ) (MPa)</td>
</tr>
<tr>
<td>( \gamma ) (MPa)</td>
<td>( c_r ) (MPa)</td>
</tr>
<tr>
<td>UCS (MPa)</td>
<td>( \phi_i ) (°)</td>
</tr>
<tr>
<td>CI</td>
<td>( \phi_{mob} ) (°)</td>
</tr>
<tr>
<td>( \sigma_t ) (MPa)</td>
<td></td>
</tr>
</tbody>
</table>

As seen in Figure 4-11, the foliation was modelled with regularly spaced elastic joints which were
rotated by 5° for every model run. The effects that the in-plane stress ratio has on breakout in anisotropic
rock was also assessed by incrementing \( K_{xy} \) between 1.25 and 2.0. It should be noted that a fully three
dimensional analysis (borehole axis considered) may yield slightly varying results to this plane-strain
model. Given the extensional nature of failure along the foliation, the evolution of stress in this axis may
not be fully developed when only assuming these conditions.
The angle of foliation changes from; (I) $\theta = 0^\circ$, initial breakout; (II) $\theta = 30^\circ$, foliation perpendicular breakout; (III) $\theta = 45^\circ$, mixed-mode breakout; and, (IV) $\theta = 90^\circ$, foliation parallel breakout.

As foliation rotates between $0^\circ$–$90^\circ$ relative to $\sigma_H$, there are three distinct types of breakout that are seen. The first occurs between $0^\circ$–$35^\circ$, where breakout is roughly perpendicular to the foliation. It can be seen in Figure 4-11, that for an increasing $K_{xy}$, the breakout azimuth tends to be dictated less by foliation orientation. This can be expected given the increasing magnitude of tangential wall stress, which governs breakout in homogenous rock. This stress dependency in foliated rock is well exhibited in the results, where breakout is less extensive for increasing stress ratios, and appears to rotate back towards the $\sigma_H$ axis. As the discrepancy between foliation angle and the principal stress axis increases to $45^\circ$, breakout begins to occur both perpendicular and parallel to foliation between the individual planes of weakness. In low
stress ratios, this sometimes creates four distinct breakouts around the circumference of the borehole. This results in a large increase in the opening angle as the four breakouts begin to merge into a larger degree of borehole failure.

Once the foliation orientation approaches $90^\circ$ to $\sigma_H$, the azimuth of breakout returns to being perpendicular with $\sigma_H$. It can be seen in Figure 4-11 that the opening angle does not return to the same size as when the foliation was parallel to the maximum principal stress. Given the inherent weakness of the joints relative to the intact rock, this result should be expected. It may, however, be somewhat dependent on the spacing of jointing and the elastic properties that were chosen.

In rhyolite, a joint set occurs parallel to foliation with the following trend (dip/dip direction):

- **RHY** — $75/217^\circ$ ($37^\circ$ foliation orientation to $\sigma_H$).

As seen in Figure 4-5, a $10^\circ$ westward rotation of stress at 1,600 m occurred within the RHY. The geometry of breakout at this point also showed a sharp increase in breakout opening angle. This supports the theory that the RHY at this location may be prone to foliation influenced breakout. It is with these types of breakout that the in situ stress could be subject to overestimation and incorrect orientation if the presence of the micro-structure in the rock is not considered, as discussed further in Chapters 5 & 6.

One limitation of the current assessment of foliation is that the change in breakout depth from foliation angle cannot be assessed given the crudely chosen spacing of structure in the model. From work done in deep oil and gas reservoirs, it has been shown that the wall strength at an anisotropy dip angle of $60^\circ$ had a minimum effective wall strength of 70% UCS at an angle to the major principal stress of $40^\circ$ (Duan and Kwok, 2015). It should however, be considered that this was work done on shale, which is associated with shear failure, rather than brittle extensional cracking.

This demonstration of the influence of foliation has not considered how the spacing of the foliation influences the results. Conceptually, more structures would act to decrease the overall strength of the material and could lead to a greater amount of breakout. This is difficult to consider with this modelling approach as it requires some form of calibration of the discontinuities with lab testing results.
Given that the joints were modelled elastically, and an inadequate number of lab tests occurred through this foliated interval, this couldn’t reliably be done. This analysis does however provide some insight into how anisotropy may influence not only the occurrence of breakout, but also excavation scale failure. In heavily foliated mining environments such as The Abitibi Gold Belt, Canada, similar excavation overbreak has been recorded (Mercier-Langevin and Hadjigeorgiou, 2011). As such, borehole failure in inherently anisotropic conditions should be an indication of expected failure mechanisms during mining development (Figure 4-12).

Figure 4-12 – Influence of anisotropy on excavation damage. Image taken from a mine in the Timmins Gold Camp (photo courtesy of Dr. K. Kalenchuk).

4.6 Breakout Summary
This chapter reviewed the techniques employed to process in situ breakout geometries from ATV surveys. This involves the selection criteria to ensure that the most representative profiles were chosen for processing and orientation, opening angle and breakout depth analysis. Based on the azimuth of breakout,
this confirms that the Major Principal Horizontal Stress for the study area is approximately East-West, which is confirmed by logging of disking geometry and compares well with regional measurements of stress in the Sudbury Basin and more broadly throughout the Canadian Shield (Herget, 1987).

The occurrence of anisotropy in the rock was also explored as it relates to the pervasive foliation in the rhyolite lithology. Modelling was conducted to review how the orientation and shape of breakout may be affected by such features in the rock, where it was seen that the orientation of breakout preferentially aligns normal to the strike of the foliation. This was particularly apparent in model runs where the in-plane stress ratio was closer to unity.

From both pilot holes 467 profiles were processed for use in estimating stress. This represents a rejection rate of almost 60% of all observed breakout profiles in situ. The following chapter discusses the implementation of numerical modelling to create a database of breakout profiles. This is used in conjunction with the pilot hole breakout geometries to establish a continuous estimation of stress with depth (Chapter 6).
Chapter 5
Use of Numerical Back Analysis of Borehole Breakout for Creation of Modelling Database

5.1 Introduction
Since the pioneering work by Carr (1974), borehole breakout has long been recognized as an indicator of principal stress orientation in the plane perpendicular to the borehole. Since that time, much work has been done to further this knowledge into the estimation of in situ stress magnitudes. A large degree of this research has come from laboratory testing, which has found a direct correlation between the magnitude of breakout and the far-field principal stress magnitudes (Haimson and Herrick, 1985).

Despite this success, there remain large amounts of uncertainty regarding the use of borehole breakout for stress estimation. Much of this surrounds the estimated strength of the borehole wall at the time of breakout (Amadei and Stephansson, 1997), but also the manner in which current solutions of breakout consider the constitutive behaviour of the rock. This is further discussed below, with reference to previously developed analytical solutions of breakout.

5.2 Analytical Solutions of Breakout
As a first pass analysis, stress in the vicinity of a borehole in a perfectly elastic and isotropic medium can be estimated using the well-known Kirsch solution (Kirsch, 1898). This was further modified by Fairhurst (1968) to include plane-strain conditions in an already stressed earth’s crust, with the consideration of borehole and formation fluid pressure by Haimson (1968). This solution takes the form shown in Eqs. (5-1) to (5-3), with each input shown in .

\[
\sigma_r = \frac{\sigma_H + \sigma_h}{2} \left(1 - \frac{a^2}{r^2}\right) + \frac{\sigma_H - \sigma_h}{2} \left(1 - 4 \frac{a^2}{r^2} + 3 \frac{a^4}{r^4}\right) \times \cos 2\theta + \Delta \rho \frac{a^2}{r^2} \tag{5-1}
\]
\[
\sigma_\theta = \frac{\sigma_H + \sigma_h}{2} \left(1 + \frac{a^2}{r^2}\right) - \frac{\sigma_H - \sigma_h}{2} \left(1 + 3 \frac{a^4}{r^4}\right) \times \cos 2\theta - \Delta P \frac{a^2}{r^2}
\]  
(5-2)

\[
\sigma_{r\theta} = \frac{\sigma_H - \sigma_h}{2} \left(1 + 2 \frac{a^2}{r^2} - 3 \frac{a^4}{r^4}\right) \times \sin 2\theta
\]  
(5-3)

Where: 
- \(\sigma_r\) = radial stress acting at a point around a borehole
- \(\sigma_\theta\) = tangential stress acting at a point around a borehole
- \(\sigma_{r\theta}\) = shear stress acting at a point around a borehole
- \(\sigma_H\) = maximum in-situ horizontal stress
- \(\sigma_h\) = minimum in-situ horizontal stress
- \(a\) = borehole radius
- \(r\) = distance to the measurement point
- \(\theta\) = angle measured counter-clockwise from the \(\sigma_H\) direction
- \(\Delta P\) = fluid pressure difference between borehole fluid pressure and pore pressure

From this it can be deduced that the location of maximum tangential wall stress around an infinitely long borehole is located at 90° to the maximum in-plane stress direction, along the \(\sigma_h\) springline. From Eq. (5-2), it can also be seen that the maximum compressive stress around the perimeter of the opening is caused by the tangential component of stress. From this same equation it can also be derived that at high enough in-plane stress ratios, the tangential stress can exceed the tensile strength of the rock, thus creating borehole-induced tensile fractures (BITFs).

Although these equations don’t explicitly relate breakout shape to far-field stress magnitudes, they are important to consider when reviewing breakout location as they ubiquitously apply to rocks demonstrating different micromechanical behaviour.

Based on this elastic theory of stress distribution, initial work was done by Barton et al. (1988), Vernik & Zoback (1992) and Zoback et al. (1985), to estimate in situ stress from the geometry of breakout. As shown in , breakout is characterized based on its radial depth and angular extent (opening
angle). Of all the work done, the simplest approach was taken by Barton et al. (1988), who estimated the maximum principal stress in a vertical borehole with the following equation:

$$\sigma_H = \frac{C_o + \Delta P + 2P_p}{1 - 2\cos(2\theta)} \frac{1 + 2\cos(2\theta)}{\sigma_h}$$  \hspace{1cm} (5-4)

Where:

- $C_o$ = borehole wall strength (BWS).
- $P_p$ = rock pore pressure
- $\theta$ = angle measured between $\sigma_H$ and the edge of breakout.

From Eq. (5-4), it can be seen that these first methods of stress estimation were based on some prior knowledge of the stress state. In the case of the work by Barton et al. (1988), this required an assumption to be made about the magnitude of $\sigma_h$. If possible, this was done by using an estimated horizontal principal stress ratio ($K_{xy}$) from a regional stress database or by completing a costly and sometimes inconclusive hydraulic fracturing test. This method also only considers an elastic stress distribution, which does not capture the post-yield mechanics of failure. Other work done on breakout...
assumes a friction based failure criterion such as Mohr–Coulomb (Zoback, Moos and Mastin, 1985; Vernik and Zoback, 1992).

In the case of more complex behaviour such as brittle-extensional cracking, which was observed during lab testing, Diederichs (2003) and Martin (1997) have shown that initiation of failure is not confinement dependent and, therefore, is not initially governed by a substantial frictional component. For such behaviour, numerical methods are required to reach a solution, as no analytical techniques exist.

5.2.1 Breakout in Brittle Rock

In a low porosity, crystalline rock, failure initiates back from the borehole wall along trans-granular cracks that are oriented sub-parallel to the Major Principal Stress ($\sigma_H$), opening perpendicular to the minor principal stress ($\sigma_h$) axis. Similar to what is observed in uniaxial compression tests, these micro-cracks are extensional with no apparent shear offset. After a critical stress is achieved, the cracks begin to coalesce to form coherent layers parallel to the borehole wall (Figure 5-2a/b). These progressively fail into the borehole until a notch is formed, where the geometry of breakout (depth/opening angle) reaches equilibrium with the in situ stress (Haimson, 2007). This dilatant failure can be roughly approximated as spalling, which is observed in larger excavations at depth, as discussed in Chapter 2 and further in the following sections (Martin, 1997; Diederichs, 2003).

When the occurrence of breakout is compared between a granite and a rock of sedimentary origin, it can be seen that at the grain scale, the development of breakout is inherently different (Figure 5-2). Although the final dog-eared shape of breakout in each rock type is quite similar, upon closer inspection, it can be seen in the brittle granite that cracks form sub-parallel to the Major Principal Stress, which gradually flake into the borehole. In the case of the Tablerock Sandstone it can be seen that cracking forms through shear along inter- and intra-granular surfaces. This distinctly different mechanism of failure which can’t be captured in currently proposed analytical solutions.
5.2.2 Challenges of Stress Evaluation from Borehole Breakout

Given the brittle mechanics of failure of the rock types in question, the realization was made that numerical modelling would be needed to represent the proper constitutive behaviour. If each breakout profile is considered individually, the stress tensor can be iteratively changed between models until a result matching the observed breakout profile is achieved. In the context of estimating stress along the length of a borehole, this methodology would prove to be time consuming, as it requires multiple models to be run for each breakout occurrence. It also isn’t immediately apparent from the previous model results
how the stress should be adjusted to more closely approximate the observed in situ breakout. This is because the breakout geometry is inherently dependent not just on the magnitude of stress but more importantly on the in-plane stress ratio \(K_{xy}\) and the tangential wall stress ratio \(R_{\sigma}\) which, in the context of a vertical borehole, are defined as:

\[
K_{xy} = \frac{\sigma_H}{\sigma_h} \tag{5-5}
\]

\[
R_{\sigma} = \frac{3 \cdot \sigma_H - \sigma_h}{UCS} \tag{5-6}
\]

These two ratios could be adjusted between model runs until a satisfactory match is made between the model and the actual breakout profile. From the inherent dependence of each ratio on the other, by changing one and keeping the other constant, the magnitudes of stress \((\sigma_H\) and \(\sigma_h\)) would have to change. This is where difficulty arises in predicting what changes in stress have to be made between subsequent model runs to achieve the ideal result presents itself.

Walton et al. (2015) also correctly observed that this method would be inefficient at reconciling what is observed in the field with the results produced during modelling. This is particularly true in cases where breakouts have an irregular profile and cannot be re-created through homogenous, continuum based modelling. It is in this case where the approach described in Chapter 4 would need to be used, to produce an average opening angle and breakout depth for each profile along the length of the borehole. This, along with individually modelling each breakout profile would prove impractical at estimating stress along a borehole or across an entire project, where multiple boreholes would be used.

5.2.3 Proposed Approach

A proposed alternative to this is an approach creating a database of breakout profiles by iteratively adjusting the magnitude of stress around the borehole during subsequent model runs, as first shown by Walton et al. (2015). For each model, an associated breakout profile can be logged in terms of opening angle and breakout depth. Other than choosing a set of material properties which is reflective of the rock types along the length of the borehole, these models don’t yet consider the actual occurrence of breakout
in situ. Between each consecutive model, the in-plane ratio of stress \((K_{xy})\) is incremented from 1.1 to 3.0, where a greater number of models were run at ratios near unity. It was seen that the breakout geometry is highly sensitive to the change of stress ratio when it is close to 1.0, as discussed in greater detail in Section 5.4.3. For each \(K_{xy}\), the magnitude of stress is varied such that the ratio of maximum tangential wall stress to unconfined compressive strength \((R_{\sigma})\), was between 1.05 and 2.0, incremented by 0.05 for each model. This approach ideally covers the realistic bounds of stress that could exist in this tectonic setting, while interpolation using curve fitting provides continuity to the results within these bounds. From this, the observed breakout profiles in situ can be compared to the curve fits, to estimate stress from each breakout occurrence, as is further shown in Section 5.4.3 and Chapter 6.

5.3 Modelling Procedure

5.3.1 Model Setup

Creation of the breakout database was completed using the continuum based finite-difference code FLAC3D-5.0 and the embedded \(FISH\) programming language. Despite the borehole cross-section being two-dimensional, it was decided to use a 3D analysis as this provided versatility for the model code, if a full 3D perspective was eventually needed. To represent the third dimension, each model was five zones deep, with the middle zone being the point of observation of breakout. This was chosen to minimize the potential of boundary effects near the edges of the model.

A radial mesh was applied extending outwards from the borehole axis with a radius of one unit such that the observed breakout depths are normalized to the borehole size. A mesh density with the maximum side length of a zone less than 3.0% of the borehole radius was used proximal to the area of interest (0.5 radii from borehole centre). With tests on models of smaller zone size (2.0% and 2.5% borehole radius), it was found that using the proposed zone size yielded the same results as these models. Given the preference to decrease run-time and subsequently increase efficiency of the proposed technique, the 3.0% side length was opted for. Work done by Walton & Diederichs (2015) has also shown
that this zonation size is dense enough to capture the brittle failure mechanics while avoiding any mesh dependency effects.

As seen in Figure 5-3, the model geometry comprises three different regions including the borehole profile and the surrounding rock, which is divided into an outer and inner zone. This division was done to increase the zone size in the outer zone and decrease model runtime, while ensuring that no model boundary effects were observed. To further aid in run-time, an axis of symmetry in the model was used, which is the left-most boundary in Figure 5-3. This boundary, and the z-axis boundaries were fixed with respect to the out-of-plane direction but were free to move in the other two orientations. The upper and lower boundaries in the y-axis and the right-most model boundaries were fixed in all orientations to reflect the assumption that the medium extends to infinity in these orientations.

From Figure 5-3-inset, the red line represents the survey locations where each zone was monitored for changes in stress, strain and material state, over the course of the model run. The y-axis was monitored solely to ensure that no boundary effects occurred along the plane of symmetry. The circumferential line and x-axis line were formally used as the indicators of breakout dimension, as further discussed in Section 5.4.
As part of the Queen’s Geomechanics Lab’s quality assurance protocol, verification of the chosen software was done considering an elastic, Mohr-Coulomb and Hoek-Brown approach. This compared analytical solutions of stress, and results from both RS2 and FLAC3D. The results of this verification process can be found in Appendix C. A condensed version of the code used for the creation of the model database is summarized in Appendix D.

5.3.1.1 Stress Application

Stress was applied orthogonal to all boundaries with the Major Principal Stress acting in the vertical direction (y-axis) so that breakout could be observed along the horizontal axis (x-axis) and away from any model boundaries. The magnitude of stress for each model is determined based on the in-plane stress component ($K_{xy}$) and tangential wall stress ratio ($R_e$) for the given model. To solve for the in-plane stress...
components, Eqs. (5-1) and (5-2) are re-arranged and solved to consider these stress ratios and the chosen borehole strength.

The stress acting parallel to the borehole axis is often interpreted as the weight of overburden, in the case of a vertical borehole, which through much of the Sudbury Basin, has been shown to be the Minor Principal component of stress (Diederichs, 1999; Trifu and Suorineni, 2009). However, with the approach of creating a depth-independent breakout database, the overburden stress can’t be used, as this would then give a definite depth associated with each model in the database.

A set of models were run to review the influence which the out-of-plane stress ($\sigma_z$) has on the breakout shape when it is; a) the Major Principal Stress, b) the Intermediate Principal Stress and, c) the Minor Principal Stress. The results are shown in Figure 5-4, where it can be seen that breakout shape is consistent between each model run at different levels of $\sigma_z$. To maintain the non-specific nature of the models, the vertical stress was the intermediate component, taken as the average between the two other Major Principal Stress components. A constant ratio between $\sigma_z$ and $\sigma_h$ which is indicative of common values found in the Canadian Shield could have been implemented. However this approach was chosen such that regardless of the tectonic setting, the exact set of steps could be taken. For example, if this framework was applied in a normal fault (Basin and Range Province, USA) or strike-slip fault (Californian Coast, USA-San Andreas Fault) regime the magnitude of vertical stress relative to the two horizontal components are different, but could still be evaluated using the same model setup.
5.3.1.2 Core Modulus Reduction Technique

Once the model geometry is set and stresses are initialized, the simulation of borehole development is done using the core modulus reduction technique. When removing zones in FLAC3D this limits the rapid transfer of stresses to neighbouring elements. This technique involves reducing the stiffness of the zones which comprise the borehole and allowing the model to reach equilibrium. This is done over a specified number of steps where each step halves the Young’s Modulus within the specified zones. Once this process is complete, the zones are removed from the borehole and the model is allowed to reach the final state of equilibrium. In Itasca code, this is done using the “Zonk” script, which is provided.
Figure 5-5 below shows an example of the modulus reduction technique using 20 steps, and the subsequent influence on displacement, occurring along the x- and y-axis of the borehole. Each step in the profile represents the point where modulus is reduced and stress re-equilibrates. The final large step occurs when the zones are removed and final model displacement is allowed to occur.

![Graph showing displacement along x-axis and y-axis](image)

**Figure 5-5 – Displacement occurring along the x-axis (red) and y-axis (green) as a function of time (step) using the Zonk modulus reduction technique.**

To ensure that the chosen degree of zonking was adequate enough to reduce the stress reverberations within the model, a comparison between RS2 and FLAC3D was done using 20 reduction steps. As can be seen in Figure 5-6, the radial stress away from the excavation boundary in the RS2 model indicates no yield occurred. From the FLAC3D model where no modulus reduction was used, there was some yielding around the boundary in the first two zones. When the modulus reduction was used in the same model, the stress trajectory appears almost identical between the RS2 and FLAC3D models. Based on the modelling comparison, this degree of core softening was carried through for the remainder of the breakout modelling.
5.3.2 Constitutive Model Considerations

Within the field of geomechanics, the modelling of rock mass behaviour is often done either based on the linear Mohr-Coulomb or non-linear Hoek-Brown failure criterions. In each of these cases, it is assumed that both the cohesion and frictional portions of strength are synchronously mobilized and are therefore additive. Even if residual values for strength are used in a conventional strain-softening approach, the components of strength reach their maximum upon yield, at which point they decrease during the post-peak range. Given the inherent nature of brittle failure, as discussed in Chapter 2, it has been shown that conventional modelling approaches are unable to capture the extent of failure in such rock conditions (Hajiabdolmajid, 2001).

5.3.2.1 Implications on Brittle Material Behaviour

The first recognition that some geomaterials may exhibit distinct strength components was by Schmertmann (1962), from work in stiff soils. He demonstrated that both the cohesional and frictional components of strength were not simultaneously mobilized, with cohesion being at its peak early in a test, while the frictional component didn’t attain its final mobilized state until 10-20 times more strain. This led to the expansion of these results into the realm of brittle mechanics by Martin & Chandler (1994),
who showed that the frictional strength fraction of a granite is only mobilized after a large portion of the rock’s cohesion has dissipated. This is intuitively satisfying, as during brittle failure, the initiation of cracking is dilatant in nature and thus only exceeds the cohesive strength of the material. Once damage occurs, these cracks are able to displace in shear which marks the point where frictional strength begins to influence the rock’s strength. When significant crack interaction has occurred and deformation takes on a more shear dominated mode, the frictional strength assumes a more dominant role, as exhibited in Figure 5-7. This therefore requires a strain based failure criterion, to define the development of each component of strength throughout damage accumulation (Hajiabdolmajid, Kaiser and Martin, 2002).

Figure 5-7 – Demonstration of the weakening of cohesion and friction in a brittle material as compared to a) laboratory compression tests and, b) underground excavations. $c_i$ and $c_r$ represent the initial and residual cohesion and $\varepsilon_c$ and $\varepsilon_f$ are the components of plastic strain, where the cohesion and friction are at the final values (Hajiabdolmajid, Kaiser and Martin, 2002).

5.3.2.2 Chosen Material Properties

Given the brittle nature of the rock under consideration, the cohesion-weakening-friction-strengthening model was used. From the generalized nature of the model database, a set of average material properties for the lithologies were selected as a base input for the CWFS model. This has been summarized in Table 5-1, in terms of the average lab properties and their translation into functional inputs for modelling.
As the rock strength was the input which is most variable between each rock type, this supports the decision to normalize the tangential wall stress to the borehole strength. This allows, in theory, for the different rock types to be assessed in parallel, despite their inherently different strengths. If this wasn’t the case, the proposed methodology would be impractical given the amount of models that would need to be run and interpreted for each lithology and associated rock strength. This approach of normalizing borehole strength across all rock types is shown in Section 5.3.3, where models from various stress states and associated strengths are shown to have similar failure profiles. Given these results, confidence exists in choosing average strength to represent the various lithologies through the normalized stress ratio.

Most of the parameters used in the CWFS model also do not vary significantly in brittle rock, with the exception of the initial cohesion, given its dependence on rock strength. This further reinforces the decision to normalize the stress ratio to the borehole strength. Based on work by Diederichs (2003, 2007), it was shown that the initial friction angle ($\phi_i$) of brittle rock is within the range of $10^\circ$ to $20^\circ$. For the breakout analysis, it was therefore chosen to use a moderate value of $15^\circ$. From research by Diederichs (2003) and Martin (1997) it has been shown that the short-term in situ strength of rock is 70% to 80% of the UCS, which is broadly described as the crack-damage (CD) threshold. Given that the CD was measured throughout lab testing, this was used as an input for the modelling database. Taking the average crack damage across all lithologies of 170 MPa, the initial cohesion ($c_i$) is expressed by the following equation:

$$c_i = \frac{BWS \cdot (1 - \sin \phi_i)}{2 \cdot \cos \phi_i}$$  \hspace{1cm} (5-7)$$

where $\phi_i$ is the initial friction angle and $BWS$ is the borehole wall strength.

The residual cohesion ($c_r$) was set equal to 5% of the initial cohesion, as proven by the back analysis of excavation overbreak by Hajiabdolmajid et al. (2002) and Diederichs (2007). This same work also suggests an appropriate range of mobilized friction angle ($\phi_{mob}$) between 45° and 65°. Based on the observations that a higher value is more representative of a very brittle and homogeneous rock type, 55° was used throughout modelling of this faintly to moderately foliated rock.
The choice of dilation angle ($\Psi$) was made based on the work by several authors. Walton & Diederichs (2015) have shown that, from back analysis of pillar yield in Sudbury Granitoid, the estimated dilation angle after yield was approximately 55% of the peak friction angle. Given the fact that during the formation of breakout or throughout underground excavation only a fraction of yielded material remains in place providing confinement, this may suggest that the dilatant component for this application should be less than what is indicated from that work. From work by Edelbro (2009), it was concluded that for hard rock masses, the dilation angle should be assumed to be between 0°-20°. This is further supported by Hoek & Brown (1997), who stated that for very good quality rock masses a dilation angle equal to 25% of the peak friction angle should be adopted.

More recently, work by Alejano & Alonso (2005) and Walton & Diederichs (2015) have shown the confinement and plastic strain dependency of dilation. For estimating a constant dilation angle to account for this, Alejano & Alonso (2005) proposed a peak dilation angle that considers the peak friction angle, rock strength and level of confinement, as seen in Eq. (5-8). For consideration in this work, the peak friction angle of 55° was used while the rock strength was varied between 160-240 MPa and confinement between 0-10 MPa. Taking an average across these ranges, this produced an estimated dilation angle of 19.5° or 35% of the peak friction angle. Walton & Diederichs (2015) proposed a similar constant dilation angle estimation strategy involving the rockmass strength ($\sigma_{crm}$), elastic tangential wall stress ($\sigma_{e_t}$) and friction angle (Eq. 5-9). By varying the ratio of strength to stress between 0.5 and 1, corresponding to a $R_s$ from 1 to 2, this produced an average dilation angle of 9.8° or 18% of the peak friction angle. From these several estimates of dilation angle, a conservative value corresponding to 20% of the peak friction angle has been chosen. This has been done acknowledging the potential variability of dilation angle, which is further investigated during the sensitivity analysis in Section 5.3.3.

$$
\Psi_{peak} = \frac{\varphi_{peak}}{1 + \log(UCS)} \cdot \log \frac{UCS}{\sigma_3} + 0.1
$$

(5-8)
\[
\frac{\psi}{\varphi} = \left( \frac{\sigma_{crm}}{\sigma_{et}} \right) - 0.1
\]  \hspace{1cm} (5-9)

It should be noted that for cohesion and friction angle, each reached final values at identical magnitudes of plastic shear strain of 0.2% \( \text{\textit{eps}}_{\text{mob}} \). Although authors such as Hajiabdolmajid, Kaiser and Martin (2002) have shown that cohesion may reach a fully residual state earlier than friction in some brittle rocks, without in situ monitoring data, it was decided to synchronize these rates.

**Table 5-1 – Strength parameters used for CWFS material model for the Victoria Project geology.**

<table>
<thead>
<tr>
<th>( E ) (GPa)</th>
<th>( \gamma )</th>
<th>( c_i ) (MPa)</th>
<th>( c_r ) (MPa)</th>
<th>( \varphi_i ) ((^\circ))</th>
<th>( \varphi_{\text{mob}} ) ((^\circ))</th>
<th>( \text{\textit{eps}}_{\text{mob}} ) (%)</th>
<th>( \Psi ) ((^\circ))</th>
<th>( \sigma_t ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>86.8</td>
<td>0.25</td>
<td>54.3</td>
<td>2.7</td>
<td>15.0</td>
<td>55.0</td>
<td>0.2</td>
<td>11.0</td>
<td>13.5</td>
</tr>
</tbody>
</table>

The strength envelope in principal stress space can be seen in Figure 5-8, which is based on the Damage Initiation and Spalling Limit (DISL) approach by Diederichs (2007). From the average laboratory results, this produces the peak and residual strength envelopes as shown, with the initial peak strength being governed almost exclusively by the cohesive component. As plastic shear strain accumulates, this envelope gradually progresses to the residual profile, which is largely dictated by friction.
Figure 5-8 – The strength envelope of the average rock properties from the Victoria Project. The green line represents the initial strength envelope, which is largely governed by cohesive strength, while the red line is the ultimate strength envelope from the mobilized frictional strength.

5.3.3 Model Input Sensitivity
The generalized approach taken when selecting material properties for the modelling database has the potential to under- or overestimate stress across each lithology. Due to this, it was decided to complete a sensitivity analysis of all inputs from laboratory testing and ones assumed from literature. This provides a basis to quantify whether averaging material properties along the length of the borehole may have a measurable effect on the final estimate of stress. For each parameter, the sensitivity as a function of breakout shape was recorded at a constant in-plane stress and tangential wall-stress ratio of 1.5. In all cases the model’s breakout shape was determined with respect to the reversal of volumetric strain, which is used as the indicator of breakout throughout the thesis (discussed in Section 5.4.2).
5.3.3.1 Lab-Based Material Inputs

Of the material properties determined from lab testing, the crack damage threshold, tensile strength and Young’s Modulus were examined to determine sensitivity of the assumed inputs. Given the observed range between lithologies of each parameter, this was important to validate the averaging of values as inputs to the modelling database.

5.3.3.1.1 Normalized Strength

As previously described, the tangential wall stress was normalized to assumed borehole strength, due to the variation seen during lab testing. For the model input of strength, this took the average crack damage threshold from all rock types. Despite there being over 80 MPa in difference in crack damage threshold across the lithologies, by normalizing the stress to strength, in theory, this allows rocks of varying competency to be assessed in parallel. Prior to creating the database, this theory was tested to ensure reliable stress results could be derived from the same set of models.

The process of validating the normalized strength approach involved running a set of models of identical material properties other than the borehole wall strength (BWS) and initial cohesion. As previously discussed, these two are proportional and therefore were changed in unison between each model. Rock strength was varied at 25 MPa intervals between 100-300 MPa, where each model applied a stress proportional to the strength. The result of this analysis is shown in Figure 5-9.
Figure 5-9 – Sensitivity analysis results of assuming the normalized model strength approach, with consideration of change in breakout geometry, as defined by the opening angle and normalized depth. Range of borehole wall strength is from lab testing of intact crack damage threshold values, with all changes in shape referenced to the 200 MPa model.

From the results of varying borehole wall strength, it can be seen that the opening angle is more sensitive to changes in assumed strength/stress in the model. Despite this, within the strength range, both shape indicators have less than ±5% error from the reference 200 MPa case. Given the argument proposed by Amadei and Stephansson (1997), that in a perfectly homogeneous and isotropic rock, stress measurement error is between 10-25%, the deviation from the assumed value falls within this range.

In the lower ranges of strength (100-150 MPa), there is a notable change in breakout shape. This may be due to the fact that, at these strengths, the other assumed material inputs aren’t representative of the material’s behaviour. Cai (2010) demonstrated that the Hoek-Brown parameter for strength, \( m_i \), is approximately the ratio between compressive and tensile strength. As a result, it was also stated that it can
be used as an indicator of the rock brittleness. This is intuitively satisfying as materials with low compressive strength would be thought to fail preferentially through shear before building up enough resistance to fail as a result of tensile induced cracking. According to this theory, for a rock to be considered brittle, it should have an associated strength ratio of 15-20, although in some cases it has been shown that brittle mechanisms occur at ratios as low as 10 (Cai, 2010). From lab testing the tensile strength ranged from 12.7-14.6 MPa, which at these lower assumed borehole strengths, gives an estimated $m_i$ from 6.8-11.8.

If these upper- or lower-threshold strengths exist along the length of the borehole in question, it is advisable that a separate set of curves be used when estimating strength in these ranges, given the inherently different mechanical behaviour at these points. This may necessitate different assumptions of input values for the CWFS approach or the consideration of a different failure criteria, such as the Hoek-Brown, which is better suited at lower $m_i$ values.

5.3.3.1.2 Tensile Strength
To evaluate the influence of tensile strength on breakout shape, models were run from 10 MPa to 17 MPa at an interval of 1 MPa. As previously described, the average tensile strength from each lithology in lab testing ranged from 12.7 (RHY) to 14.6 (MTGB). The results in Figure 5-10 demonstrate that the overall shape of breakout changes very little as a function of tensile strength, with the opening angle showing a slightly greater variance than breakout depth. This same result was achieved by Edelbro (2010), who showed that this input had virtually no influence on excavation damage size.
Figure 5-10 – Sensitivity analysis results of tensile strength, with consideration of change in breakout geometry, as defined by the opening angle and normalized depth. Range of strength is from lab testing, with changes in shape referenced to the 13 MPa model.

5.3.3.1.3 Young’s Modulus

The rock stiffness was considered by running models from 60-100 GPa at an interval of 10 GPa. Similar to the tensile strength, this parameter showed little influence on the overall shape of breakout, with the opening angle being more sensitive at a lower stiffness, while breakout depth was slightly more sensitive near the upper-bound. Given the observed lab testing range between 71-97 GPa, there is virtually no change between these bounds, as seen in Figure 5-11.
Figure 5-11 – Sensitivity analysis results of Young’s Modulus, with consideration of change in breakout geometry, as defined by the opening angle and normalized depth. Range of stiffness is from lab testing, with all changes in shape referenced to the 85 GPa model.

The use of average material values across the modelling database appears to have little influence on the overall result of breakout shape. It is however suggested that in each specific project where this modelling approach is used, a separate sensitivity analysis be completed with the respective material properties. Although the ranges of interest at Victoria Project fit within reasonable bounds of error, it can be seen, for example, if the ratio of borehole strength to tensile strength decreases below a certain value, the process of averaging strength may result in significant errors in stress estimation.

5.3.3.2 CWFS Material Assumptions

For the implementation of the cohesion-weakening-friction-strengthening constitutive model, a number of inputs were chosen based on published results in similar rocks. This included the residual cohesion, initial/mobilized friction, dilation and rate of strength mobilization. The sensitivity of each was assessed within valid ranges from literature. In the case of peak cohesion, this was not assessed as it is intimately
related to strength and initial friction angle, and therefore is considered during the borehole wall strength normalization.

5.3.3.2.1 Residual Cohesion

For the creation of the modelling database, residual cohesion was set to 5% of the peak cohesion. This was shown to be appropriate by Hajiabdolmajid et al. (2002) and Diederichs (2007) based on the back analysis of excavation overbreak. For the sensitivity analysis, a range from 0%-10% of peak cohesion was assessed. As can be seen in Figure 5-12, the breakout depth appeared to be the most sensitive to changes in cohesion, although generally this input had little effect on breakout shape over this range. The most sensitive location was when residual cohesion was set to zero, where both the opening angle and breakout depth show a notable increase.

![Figure 5-12](image-url)

**Figure 5-12 – Sensitivity analysis results of residual cohesion as a percentage of peak cohesion, with consideration of change in breakout geometry, as defined by the opening angle and normalized depth. All changes in shape are referenced to the 5% model.**
5.3.3.2.2 Initial Friction Angle

Based on work by Diederichs (2003, 2007), it was shown that the initial friction angle ($\phi_i$) of brittle rock is within the range of 10° to 20°. To evaluate the influence of friction on breakout shape, it was assumed that the borehole strength and initial cohesion is constant despite the intimate relationship that these three parameters have with each other. As can be seen in Figure 5-13, the opening angle shows a greater variability across the tested range with the geometry of breakout decreasing in size as a function of friction angle.

In reality, this trend is most likely less pronounced than what is shown. This is due to the influence that friction has on the initial magnitude of cohesion. This proportionality is shown in Eq. (5-10), where it is evident that as friction angle increase, the magnitude of initial cohesion decreases. Given the importance which cohesion has on the initial strength of brittle material, this would act to oppose the trend shown in Figure 5-13.

$$c_i \propto \frac{1 - \sin \phi_i}{2 \cdot \cos \phi_i}$$  \hspace{1cm} (5-10)

Figure 5-13 – Sensitivity analysis results of initial friction angle, with consideration of change in breakout geometry, as defined by the opening angle and normalized depth. All changes in shape are referenced to the 15° model.
5.3.3.2.3 Mobilized Friction Angle

Based on the same overbreak back analysis by Hajiabdolmajid et al. (2002) and Diederichs (2007), these authors suggested an appropriate range of mobilized friction angle between 45° and 65°. This is based on the observations that a higher value is more representative of a very brittle and homogeneous rock type. As can be seen in Figure 5-14, the change in opening angle and breakout depth follow an opposite trend. Other than the dilation angle analysis, this trend wasn’t observed in the other material property sensitivity analyses, although it was similarly reported by Edelbro (2010). The change in opening angle was much more pronounced than the breakout depth over the analyzed range with up to a 25% change in predicted breakout. Given this elevated sensitivity, the mobilized friction angle should be carefully considered when implementing the CWFS constitutive model.

![Figure 5-14](image)

**Figure 5-14** – Sensitivity analysis results of mobilized friction angle, with consideration of change in breakout geometry, as defined by the opening angle and normalized depth. All changes in shape are referenced to the 55° model.
5.3.3.2.4 Dilation Angle

Based on the discussions in Section 5.3.2 on dilation angle selection, a range from 0-20° was considered in the sensitivity analysis. Similar to the mobilized friction angle, the change in depth and opening angle showed the opposite trend. At lower dilation angles breakout depth was increased, while larger dilation angles showed a pronounced increase in opening angle (Figure 5-15). Given this level of sensitivity of the input, it should also be carefully considered during the modelling process.

![Figure 5-15 – Sensitivity analysis results of dilation angle, with consideration of change in breakout geometry, as defined by the opening angle and normalized depth. All changes in shape are referenced to the 12.5° model.](image)

5.3.3.2.5 Rate of Strength Mobilization

Given the lack of data, the rate of strength mobilization between friction and cohesion was synchronized at 0.2% for the creation of the modelling database. The influence that this could potentially have on the shape of overbreak was assessed between a range of 0.1% to 0.4% plastic shear strain. A mixed-shear strain case was also considered where cohesion reached its residual value at 0.2% while friction wasn’t
fully mobilized until 0.5%. This was the chosen input of plastic shear strain from the back analysis of the Lac-du-Bonnet Granite by (Hajiabdolmajid, Kaiser and Martin, 2002).

From Figure 5-16, it can be seen that the rate of strength mobilization has a significant impact on opening angle, but less so on breakout depth. Based on the fact that breakout is overestimated at lower plastic shear strain values, this is an indication that cohesion is the dominant input governing the rock behaviour. This trend continues beyond the reference point (0.2%) where the breakout is then significantly underestimated after this point. For the mixed strain case, this produces an overestimate of breakout which denotes a weaker material. Similar to the initial/mobilized friction angle and dilation angle, this input into the CWFS should be carefully considered and supported during modelling.
5.3.3.3 Sensitivity Analysis Summary

During the sensitivity analysis, material property inputs were assessed to determine their relative influence on the size of breakout. This includes the consideration of the averaged material properties from lab testing and assumptions made based on literature. Although the results presented in Table 5-2 can be considered a proxy to determine the sensitivity of the future stress result, they aren’t fully proportional. This is evident upon inspection of the governing equations of breakout in Section 5.4.3, where the relationship between shape and stress take on a highly non-linear form. In the case where both opening angle and breakout depth increase/decrease synchronously, then it is clear that the estimate of stress will also change accordingly. It becomes more complicated in cases such as the change in dilation angle, where angular extent and depth show an opposite trend in shape change. This is where a more detailed comparison needs to be done, as discussed in Chapter 7.

From Table 5-2, it can be seen that the assumption of averaging material properties from lab testing showed little influence on the extent of breakout over the range of interest. The same thing can’t be said of the material properties which make up much of the inputs for the CWFS approach. Of these, the magnitudes of initial and mobilized friction appeared to result in the greatest change. It is recommended when adopting this constitutive model, that each of these parameters are carefully considered and supported by literature in similar rock types.

Table 5-2 – Summary of sensitivity analysis of material properties on breakout shape.

<table>
<thead>
<tr>
<th></th>
<th>BOA</th>
<th>NBD</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Borehole Strength</td>
<td>4.0%</td>
<td>2.5%</td>
<td>Shape increase with normalized strength</td>
</tr>
<tr>
<td>Tensile Strength</td>
<td>3.0%</td>
<td>0.5%</td>
<td>No noticeable change</td>
</tr>
<tr>
<td>Young's Mod.</td>
<td>3.5%</td>
<td>2.0%</td>
<td>Shape increase with decrease in stiffness</td>
</tr>
<tr>
<td>Residual Cohesion</td>
<td>18.0%</td>
<td>9.0%</td>
<td>Shape increase with decrease in cohesion</td>
</tr>
<tr>
<td>Initial Friction</td>
<td>22.5%</td>
<td>7.0%</td>
<td>Shape increase with decrease in friction</td>
</tr>
<tr>
<td>Mobilized Friction</td>
<td>25.0%</td>
<td>9.0%</td>
<td>Opposite trend in shape change between depth and opening angle</td>
</tr>
<tr>
<td>Dilation</td>
<td>16.5%</td>
<td>6.5%</td>
<td>Opposite trend in shape change between depth and opening angle</td>
</tr>
<tr>
<td>Strength Mobilization</td>
<td>19.0%</td>
<td>1.5%</td>
<td>Shape increase with decrease in plastic shear strain</td>
</tr>
</tbody>
</table>
5.4 Evaluation of Breakout and Damage Zone Profiles

From each model run, the opening angle and normalized depth of breakout had to be assessed. As the models were built with a borehole radius of 1, they could then be scaled to also predict the amount of excavation overbreak at the size of the shaft, provided the proper strength assumptions were made. The following work outlines the important model parameters which were monitored for the determination of both borehole breakout and excavation overbreak. By compiling the results from each model, surfaces were fit through the results to provide a continuous estimate of breakout/overbreak.

5.4.1 Concept of Excavation Zonation

The concept of damage zonation during excavation was first conceived for the purpose of underground nuclear waste repositories in the early 1980’s (Kelsall, Case and Martin, C Dhabannes, 1984). It was done in recognition of the fact that, as distance from an excavation increases, the intensity of damage and subsequent permeability decreases. As a collection, the excavation damage zones are referred to as the EDZs, with the various zones illustrated in Figure 5-17. Broadly speaking, the damage accumulated around an excavation can be divided into two categories; inevitable stress induced damage and damage as a consequence of the construction method (CDZ). The latter of these will not be considered in this work as the extent of this zone is a direct result of the excavation method used and can theoretically be reduced to zero if care is taken (Perras and Diederichs, 2016).

Proximal to the excavation, the highly-damaged zone (HDZ) is denoted by the interconnection of macro-structures with very little remaining confining stress (Figure 5-17). More distal from the HDZ is the transition to the EDZ which is sub-divided into EDZ, and EDZ, with the inner having remaining interconnection of induced structure with only isolated damage towards the outer boundary (EDZ,). All damage within the EDZ is irreversible with a decreasing amount of dilation from the EDZ, towards the EDZ,. The outermost zone is the excavation influence zone (EIZ) which represents the elastic region of deformation where measurable deviation is seen from the far-field stress state. For the purpose of the
estimation of shaft overbreak, the extent of EIZ will not be considered as it mainly is of concern when reviewing the influence of a varying stress on surrounding infrastructure and excavations.

![Diagram](image.png)

Figure 5-17 – The occurrence of excavation damage zonation showing a) the induced crack intensity (Perras 2014, after ANDRA 2005) and, b) the relative geometry of each zone and effect of discontinuities (Perras, 2014).

5.4.2 Model EDZ Indicators

Based on the modelling results presented herein, it was found that plastic shear strain \((\varepsilon_{PS})\), volumetric strain \((\varepsilon_v)\), yielded elements and minimum principal stress \((\sigma_3)\) were the best parameters for determining the extent of each EDZ (Figure 5-18/Figure 5-19). This is supported from previous work done by Perras et al. (2012) and Perras & Diederichs (2016). For the purpose of determining EDZ depths and opening angles, the same indicators were used, however the criteria varied slightly between the two. Each EDZ transition depth (Figure 5-18a) was determined as follows:

1) **HDZ-EDZ** \(_i\): the point at which \(\sigma_3\) begins to increase from its level at the excavation surface

2) **EDZ\(_i\)-EDZ\(_o\)**: the point at which volumetric strain becomes tensile (expansion to compression boundary).

3) **EDZ\(_o\)-EIZ**: the point at which plastic strain begins to accumulate, as confirmed by both the yielded elements profile and plastic shear strain.
The opening angle of each EDZ (Figure 5-18b) took the following form:

1) HDZ-EDZ\textsubscript{i}: the first point around the circumference at which $\sigma_3$ is tensile (Figure 5-18b-inset)

2) EDZ\textsubscript{i}-EDZ\textsubscript{o}: the point at which volumetric strain and plastic shear strain show a sharp uptake before gradually decreasing to the final extent of yielding.

3) EDZ\textsubscript{o}-EIZ: The point at which plastic strain begins to accumulate, as confirmed by both the yielded elements profile and plastic shear strain.

The theory behind the use of these metrics begins with the consideration of plastic yielding. At any point surrounding the excavation where plastic strain is deemed to have occurred, it is a result of the exceedance of the rock’s strength envelope. This denotes the onset of damage within the rock, which begins at the EDZ\textsubscript{o}-EIZ boundary. Upon review of Figure 5-18a, it can be seen that at this point volumetric strain is still contracting, indicating that despite some level of damage there is still active confinement inhibiting the growth and propagation of grain-scale fracturing.

Where volumetric strain switches from compression to extension (EDZ\textsubscript{i}-EDZ\textsubscript{o} boundary) it indicates in a real rockmass that damage can coalesce at a grain-size scale. As can be seen in Figure 5-18, this boundary is also seen as a change in stress accumulation rate in the case of both failure depth and opening angle. This is evident in the maximum shear stress and maximum principal stress profiles. For a brittle material, this increase in volumetric expansion with decrease in stress would cause the formation of macro-scale fractures (Perras and Diederichs, 2016). Based on this theory, the EDZ\textsubscript{i} limit will also be considered as the maximum extent of potential borehole breakout. This is intuitively satisfying as at this point, fractures have interconnection but limited displacement. Despite this, during the drilling process, the presence of water and vibrations act to further propagate cracking, which causes the extent of the EDZ\textsubscript{i} to fail into the hole after a period of time, leaving the final breakout profile.
Figure 5-18 – Numerical results for $K_{xy} = 1.7$, $R_\sigma = 1.8$ with determination of EDZs/breakout geometry in terms of a) normalized depth and b) opening angle. Inset from b) includes enlarged plot of minimum principal stress.
Figure 5-19 – Modelling results for $K_{xy}=1.7$, $R_{σ}=1.8$ with highlight of EDZs/breakout geometry.

*Interpretation of each boundary can be seen above in Figure 5-18.*

The definition of the HDZ boundary differs slightly between the depth and opening angle. In terms of failure depth, when $σ_3$ begins to increase away from the excavation, this denotes the point where the rock is under some confinement, which limits macro-scale fracture growth. Anywhere within the HDZ-EDZ$_i$ transition is unconfined, showing more significant accumulated strain. In the case of the HDZ opening angle, this was deemed to be the first location where $σ_3$ is tensile. This can be seen in Figure 5-18a-inset, where there is a small zone of tension around the circumference. Similar to the HDZ failure depth, at this point, the rockmass is fully unconfined and as a result experiences tensile stress. It should also be noted that the EDZ$_i$-EDZ$_o$ transition for opening angle was often coincidentally at the location of maximum tensile stress around the excavation’s circumference. This relation and the proximity of the HDZ and EDZ$_i$ limits was particularly apparent at high in-plane stress ratios. An example of this is shown in Figure 5-19, where the HDZ and EDZ$_i$ angular extents are almost the same at the borehole boundary. The selection of these limits is shown in the inset plot of Figure 5-18, where the extent of tensile stress on the excavation wall is only $4^o$ wide and comprises both envelopes. This leads the author to believe that
these two EDZs converge near the excavation boundary and could be treated as one location. This is intuitively satisfying given the nature of the confinement dependency of brittle failure (Diederichs, 2007). Near the surface of the excavation, cracks are able to more easily propagate and coalesce, thus leading to large amounts of strain and failure, which is indicative of the HDZ profile. It is this relatively quick transition from crack formation, propagation and ultimate yield at the excavation wall, that causes the angular extent of the profiles to be similar. This is particularly the case at high in-plane stress ratios where elevated deviatoric stress leads to greater amounts of damage in the rock (Martin, 1997).

5.4.3 Modelling Database Parameters

In creating the modelling database, 160 models were characterized based on failure geometry for each EDZ type and breakout profile. From this, surfaces were fit relating the in-plane stress ratio and normalized tangential wall stress to the breakout/overbreak geometry (Figure 5-20). This was done using the curve fitting software Table Curve 3D, that provided a set of possible fits to the given data. This included a range from simple polynomials to more advanced functions such as Chebyshev rationals. For each function type, goodness of fit is established through minimization of least squares to obtain the appropriate constants. In each case an $R^2$ value is reported and was the basis for the selection of the equations to represent both the opening angle and breakout depth as a function of the stress ratios.

The selection of the representative equations was a balance of having the best fit to the data as possible, while limiting the complexity of the respective equations. In some of the more complicated functions, although they had extremely close fits (>99%), they were quite cumbersome and, often were “over-fits” to the data. In Figure 5-20, the fits to the EDZ$_i$ are shown, where each point represents a measurement from one of the models, and the surface is the fit to the entire dataset. From the generalized functions shown below, these were used on the other HDZ and EDZ$_o$ results, where closeness of fit was optimized in Excel through the same least squares method. A plot of residuals in each case was created to also ensure that the functions weren’t over-fitting based on the observed trends, as seen in Figure 5-21.
Figure 5-20 – Original fit surfaces for the EDZ$_{\alpha}$, showing model results (points). Colour of point outline shows whether the function is greater than (red) or less than (blue) the measured model value at each point. The colour inside each point is the closeness of fit gradationally changing from green (exact match) to red (poor match).

Normalized Breakout Depth

\[
NBD = \frac{a_2 + b_2 \cdot \ln(K_{xy}) + c_2 \cdot \ln(R_\sigma - d_2)}{1 + e_2 \cdot \ln(K_{xy}) + f_2 \cdot \ln(K_{xy})^2 + g_2 \cdot \ln(R_\sigma - d_2)}
\]

\[R^2 = 0.98\]

Breakout Opening Angle

\[
BOA = e^{\left(a_1 + b_1 \frac{\ln(K_{xy})}{K_{xy}} + c_1 \frac{\ln(R_\sigma - d_1)}{\ln(R_\sigma - d_1)} + e_1 \right)}
\]

\[R^2 = 0.91\]
Figure 5-21 – Plot of residuals for opening angle (top) and normalized breakout (bottom) from the EDZ curve fit, showing whether the function adequately represents the model data.

For the surface fit of opening angle (BOA) and normalized depth (NBD), a minimum $R^2$ value of 91.4 and 97.7 was respectively achieved. The fits take the generalized form of:

$$ BOA = e^{(a_1 + b_1 \cdot \frac{\ln(K_{xy})}{K_{xy}} + \frac{c_1}{\ln(R_\sigma - d_1) + \frac{e_1}{R_\sigma - d_1}})} $$  \hfill (5-11)

$$ NBD = \frac{a_2 + b_2 \cdot \ln(K_{xy}) + c_2 \cdot \ln(R_\sigma - d_2)}{1 + e_2 \cdot \ln(K_{xy}) + f_2 \cdot \ln(K_{xy})^2 + g_2 \cdot \ln(R_\sigma - d_2)} $$  \hfill (5-12)

where $a_i, b_i, \ldots, g_i$ are fit constants dependent on material properties and the EDZ being considered (Table 5-3).
It should be noted that during the initial fitting process, the constant $d_i$ was not a part of the equation. This was however added, to allow for the intersection location of the curves along the x-axis to vary depending on the results from the modelling. In the case of EDZ$_o$ this occurred at $d_i=0.05$ while the EDZ$_i$ and HDZ occurred at $d_i=0.1$.

Table 5-3 – Fit constants for Eqs. 5-11 & 5-12, to create the graphical representation in Figure 5-24.

<table>
<thead>
<tr>
<th></th>
<th>HDZ $i=1$</th>
<th>HDZ $i=2$</th>
<th>EDZ$_i$ $i=1$</th>
<th>EDZ$_i$ $i=2$</th>
<th>EDZ$_o$ $i=1$</th>
<th>EDZ$_o$ $i=2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$a_i$</td>
<td>8.05</td>
<td>1.03</td>
<td>7.65</td>
<td>1.08</td>
<td>8.01</td>
<td>1.02</td>
</tr>
<tr>
<td>$b_i$</td>
<td>-4.86</td>
<td>0.10</td>
<td>-4.35</td>
<td>1.29</td>
<td>-5.41</td>
<td>0.65</td>
</tr>
<tr>
<td>$c_i$</td>
<td>-0.01</td>
<td>-0.76</td>
<td>-0.01</td>
<td>-0.38</td>
<td>-0.02</td>
<td>1.09</td>
</tr>
<tr>
<td>$d_i$</td>
<td>0.10</td>
<td>0.10</td>
<td>0.10</td>
<td>0.10</td>
<td>0.05</td>
<td>0.05</td>
</tr>
<tr>
<td>$e_i$</td>
<td>-4.67</td>
<td>0.17</td>
<td>-4.04</td>
<td>1.37</td>
<td>-3.54</td>
<td>0.72</td>
</tr>
<tr>
<td>$f_i$</td>
<td>-</td>
<td>-0.03</td>
<td>-</td>
<td>-0.03</td>
<td>-</td>
<td>-0.05</td>
</tr>
<tr>
<td>$g_i$</td>
<td>-</td>
<td>-0.88</td>
<td>-</td>
<td>-0.79</td>
<td>-</td>
<td>-0.14</td>
</tr>
</tbody>
</table>

From the curve fitting process, a number of trends were observed in the relative fit of the equations to the model results, within different stress regimes. These can be summarized as:

- A less than average fit was observed at in-plane stress ratios that were closer to unity ($K_{xy}$= 1.1).
  Model runs at a stress ratio of 1.05 were attempted although this yielded inconclusive results and was abandoned for the remainder of the modelling process. This is shown in Figure 5-22 where breakout appears to occur as two small lenses around the circumference of the borehole. From the plot of breakout depth the EDZ$_o$ is interpreted to be smaller than both the EDZ$_i$ and HDZ, thus bringing the validity of the results into question.

- Near the x-axis of each plot, the lines converge, making differentiation between each stress result difficult. This also led to less than average fits near these points as the scale of each zone within the model was not able to differentiate a change in fractions of a degree in opening angle or breakout depth. Caution should be taken when $R_\sigma$ results fit to curves which are close to the x-axis as the interpretation of in-plane stress ratio is highly dependent on the curve fits and may not be indicative of the true stress state.
• At high $K_{xy}$ the curves begin to converge such that above a ratio of 3.0, the difference in predicted overbreak is un-discernable. Because of this, these plots and equations should only be considered valid in stress regimes with a stress ratio less than 3.0.

• The predicted opening angle of all EDZs are relatively similar, this is particularly the case between the EDZ$_i$ and HDZ. For simplicity sake, a single opening angle plot could be used when predicting excavation overbreak.

![Figure 5-22 – Modelling results for $K_{xy}= 1.05$, $R_{\sigma}=1.25$ showing the location of excavation damage.](image)

The relative changes in breakout shape when the bounds of stress are considered as shown in Figure 5-23. At lower tangential wall stress ratios, this produces a smaller opening angle and breakout depth, while a lower in-plane stress ratio produces a larger breakout shape. This is not only confirmed by the highlighted models, but also by the curve fits from Eqs (5-11) and (5-12), which are shown in Figure 5-24. At lower $R_{\sigma}$ this results in a greater overall volumetric strain while at higher $K_{xy}$ the volumetric strain is decreased by an order of magnitude. This can be explained by the fact that at lower in-plane stress ratios, to satisfy the required tangential wall stress ratio, this requires greater overall magnitudes of model stress, thus leading to greater strain.
Figure 5-23 – Model results showing the extremes of breakout profiles based on in-plane stress ($K_{xy}$) and tangential wall stress ($R_z$) ratios.

Figure 5-24 is a summary of the curve fits for each overbreak/breakout envelope, with the third dimension presented as each line on the plot. For the graphical determination of stress or overbreak geometry, these plots are used as nomograms to derive a solution based on the in situ breakout profiles from both pilot holes. Alternatively, Eqs. (5-10) and (5-11) can be solved to find a unique stress solution that produces the same opening angle and depth for each observed breakout. The procedure for either the numerical or graphic solution, associated considerations and stress/overbreak results are outlined in Chapter 6.
Figure 5-24 – Plots from Equations (5-11) and (5-12) using appropriate fit constants from Table 5-3, for use in graphical determination of stress and overbreak. Each line defines an in-plane stress ratio-$K_{xy}$, which is shown along the border of each plot.
5.5 Conclusion

From the lab testing completed as part of this dissertation, it was evident that the rock types in question failed through brittle-extensional cracking. Due to this, no currently developed analytical solutions of breakout could be used when assessing stress. The choice was made to use a cohesion-weakening, friction-strengthening approach, which has been shown by a number of authors to replicate the extent of brittle failure (Hajiabdolmajid, Kaiser and Martin, 2002; Diederichs, 2007; Edelbro, 2010). During this process, material properties were either chosen as an average from lab testing results or were chosen from supported literature in similar rock types. To validate these assumptions, a sensitivity analysis was completed which showed that the averaging of material properties from lab testing had less than a 5% influence on breakout depth and opening angle. In the case of CWFS assumptions such as friction angle, cohesion, dilation angle and strength mobilization rate, these all had a more pronounced influence on breakout shape. This was in the order of 15-20% in most cases, with the initial and mobilized friction angles being the most sensitive. Due to this, it is recommended that a detailed review of literature be completed prior to selecting these inputs for modelling.

The process of evaluating breakout and excavation overbreak was made with respect to the concept of excavation damage zonation (EDZs). This included the model indicators of each damage zone and the curve-fitting procedure to produce an estimate of opening angle and breakout depth with respect to in-plane stress and tangential wall stress ratios. This produced the curves for the highly damaged, inner damage and outer damaged zones, as shown in Figure 5-24. From these plots, the process of estimating stress is shown in Chapter 6 and excavation overbreak prediction in Chapter 7.
Chapter 6
KGHM Victoria Stress Profile

6.1 Introduction

Upon creating the generalized breakout curves discussed in Chapter 5, the in situ breakout results from both pilot holes are now used for the final interpretations of in situ stress state. Throughout this process, and in the previous two chapters, a number of key assumptions were outlined and justified. These include:

- Observed breakout is not influenced by macroscopic rock structure such as joints or healed fractures.
- The time-dependency of breakout is accounted for by the elapsed time between the survey and hole completion.
- Numerical model inputs from lab testing that include; Young’s Modulus, Poisson’s ratio, tensile strength and unconfined strength, can be averaged across the lithologies.
- The assumption of material property inputs for the CWFS modelling approach not from lab testing, that include; the initial/mobilized friction angle, dilation angle, residual cohesion and plastic shear strain increment. These were chosen based on published literature.

Previous work on this topic is discussed herein, which guided the assumptions made in the analysis. The sensitivity of the results to the presence of borehole fluid was also explored to see how it affects the estimation of stress.

In the case of the prediction of stress, both pilot holes were compared to see the closeness of stress magnitude and in-plane stress ratio prediction, and most importantly to see whether each demonstrates similar trends in stress changes. Pilot hole GT0020VCa stress results were also compared against the depth profile of strength and stiffness parameters from lab testing. This was done to relate any notable deviations in material properties to an observed change in predicted stress. Throughout the water sensitivity and material property review, borehole FNX-1204 was not considered, given the previously
discussed data quality concerns and the greater relevance of GT0020VCa due to its alignment along the current shaft axis.

6.2 Effective Borehole Wall Strength
As discussed in Chapter 5, research in the field of geomechanics has shown that the effective in situ strength of rock at the scale of a borehole or excavation is not equal to its peak lab strength. This can mostly be explained by the fact that in laboratory tests, samples experience one continuously applied loading cycle, while in situ, the evolution of stress is much more complicated than this. Unlike in laboratory based strength tests, where a sample is monotonically loaded, the progression of spalling failure is highly dependent on the stress path taken throughout the excavation process. This path involves stress increases, decreases and rotation, which begins ahead of the tunnel face or borehole, as shown in Figure 6-1 (Martin, 1997). Other factors such as temperature, humidity and the presence of heterogeneity or structure in situ also play a role in the effective strength of an opening.

Figure 6-1 – Comparison of stress path from various lab scale tests (solid line), as compared with an idealized stress path during excavation (hashed line). Each letter corresponds to a reference location during tunneling, as shown in the upper right corner (Martin, 1997).
Through the work completed at AECL’s underground research laboratory (URL) in Canada, Martin (1997) was able to show the scale and stress path effect on borehole strength. With the testing of cylindrical openings of differing diameters in a laboratory setting, the required tangential wall stress as a function of borehole size was established. This wasn’t the first time in rock that authors were able to replicate breakout at the lab scale to observe its formation. Carter (1992) showed in Borea sandstone, that at small enough diameters (less than 110 mm), the tangential wall stress required to initiate breakout is up to $2.5\text{UCS}$. From the work in the Lac du Bonnet granite by Martin (1997), this further supported these findings from tests on openings from 5-100 mm diameter. As shown in Figure 6-2, this ratio of tangential stress to peak rock strength eventually converged to 1 in laboratory tests, however it always remained higher than the values determined from in situ data at the underground lab. In the case of NQ/NQ3 core, this produced an estimated wall strength of 1.1 times the peak laboratory strength. In the case of HQ core, this ratio was approximately at unity.

From the in situ observations, it can be seen that there is a very limited scale effect on the in situ strength, with a modest effect based on the orientation of the hole relative to the stress field (Figure 6-2). When the borehole sizes in question are compared in Figure 6-2, this produced an estimate of 0.7 to 0.75 times peak strength (from in situ), which many authors have also found to be the average crack damage threshold of brittle rock (Bieniawski, 1967; Martin, 1997; Diederichs, 2007; Cai, 2010). Given the definition of $CD$ as the point where individual cracks begin to coalesce and propagate in an unstable manner, this has led authors such as Diederichs (2007) to state that $UCS$ is simply a function of the test system and sample geometry, and not that of the material condition.
Figure 6-2 – The ratio of tangential wall stress to unconfined compressive rock strength with respect to the hole diameter in Lac du Bonnet granite. Tests include lab scale-hollow points, and in situ back calculations of strength- solid points. Outer diameter of NQ/NQ3 and HQ sized core is plotted as dashed lines for reference (after Martin 1997).

Separate work has also been done considering the effects of loading time on the peak strength of UCS samples. This was summarized from literature by Paraskevopoulou et al. (2017), who showed the relative effects of testing time for metamorphic and igneous rocks. As can be seen from Figure 6-3, in both cases the sample strengths converge on the CD threshold at approximately the 4-hour mark for metamorphic rock and 30 minutes for igneous samples. This supports the assumption regarding time-dependency of breakout, as the logistical constraints of ATV surveys wouldn’t allow the profiling of the borehole prior to the degradation of strength to it’s CD threshold. Although the discussion of scale effect and loading time are separate in nature, they do however both corroborate the same result, that the upper-bound in situ strength is defined by the crack damage point.
Figure 6-3 – UCS test data for a) metamorphic and, b) igneous samples where the driving stress ratio is the applied stress at failure normalized to the average UCS from each rock type (Paraskevopoulou, Perras and Diederichs, 2017).

6.3 Stress Estimation Methodology

To determine the principal stress magnitudes from in situ borehole breakout, there are two possible approaches given the proposed modelling technique. The first is to solve Eqs. (6-1) and (6-2) for a corresponding in-plane principal stress ratio \((K_{xy})\) and tangential wall stress ratio \((R_{\sigma})\), which produces a matching Breakout Opening Angle (BOA) and Normalized Breakout Depth (NBD) for each of the borehole breakout profiles. Given the non-linearity of these equations, a simple re-arrangement can’t be
made to solve for the unknown stress state. This is where numerical methods must be used to iteratively solve such equations. This can be accomplished using Excel’s Solver add-in which has the capability of solving for the desired stress ratios, provided a proper set of bounds are placed on the solving technique. These bounds would include that $K_{xy}$ can’t be less than 1.0 or exceed 3.0, and that $R_\sigma$ must be greater than 1.0 or less than 2.0. Although in reality both these ratios could exceed the bounds which were established, the curve fitting never considered these stress states, and as such, fit quality in these regions is uncertain.

$$BOA = e^{(a_1+b_1 \cdot \frac{\ln(K_{xy})}{K_{xy}} + c_1 \cdot \frac{e_1}{R_\sigma - d_1})} \quad (6-1)$$

$$NBD = \frac{a_2 + b_2 \cdot \ln(K_{xy}) + c_2 \cdot \ln(R_\sigma - d_2)}{1 + e_2 \cdot \ln(K_{xy}) + f_2 \cdot (\ln(K_{xy}))^2 + g_2 \cdot \ln(R_\sigma - d_2)} \quad (6-2)$$

Alternatively, these ratios can also be graphically determined as shown in Figure 6-4. By arranging the modelling results in the form of a nomogram, this allows for the 3-dimensional datasets to be visually solved. The procedure to do this is as follows:

1) Draw a horizontal line at the normalized breakout depth and opening angle at the y-axis. These values are obtained from the in situ data breakout profile.

2) Trace lines horizontally from the given BOA and NBD points (from part 1) on each of the graphs.

3) Locate the tangential wall stress ratio where both horizontal lines intersect the same $K_{xy}$ curve.

4) The value of the intersected curve provides the in-plane stress ratio.

This process is particularly powerful when numerical solutions aren’t readily available for the given equations, or if stress needs to be estimated from a single breakout. Where this methodology slightly lacks is in the accuracy of the result. For example, if the lines don’t perfectly intersect at one of the curves, this means that the in-plane stress ratio is somewhere between two of them. In this case, the value for $K_{xy}$ must be chosen with some degree of error. If a streamlined work process can be established for the
numerical solution type, this also provides a far more time efficient solution when considering a dataset of breakout profiles from multiple boreholes.

Figure 6-4 – Methodology of using the nomogram approach for determining the in-plane stress ratio ($K_{xy}$) and tangential wall stress ratio ($R_\sigma$). Example from breakout in GT0020VCa at 1585 m.

Once the $K_{xy}$ and $R_\sigma$ pair have been solved for each breakout (either numerically or graphically) Equations (6-3) and (6-4), can be used to determine the magnitudes of principal stress at each location of breakout. It should be noted that in Equation (6-3) an effective borehole wall strength ($BWS$) is used in
lieu of $UCS$, which allows for the consideration of the in situ rock strength (as discussed in Section 6.2). For the purpose of this analysis, the crack damage threshold from laboratory testing of each lithology was used when resolving stress magnitudes.

$$\sigma_h = \frac{R_\sigma \cdot BWS}{3 \cdot K_{xy} - 1} \quad (6-3)$$

$$\sigma_H = K_{xy} \cdot \sigma_h \quad (6-4)$$

The presented example in Figure 6-4, was taken from a breakout profile in rhyolite at a depth of 1,585 m. The geometry of breakout had an opening angle of 42° and a normalized depth of 1.18. Using the graphical method, this gave a $K_{sv}$ of 1.3 and $R_\sigma$ of 1.44, which produces a maximum in-plane principal stress ($\sigma_H$) of 88.4 MPa and minimum in-plane principal stress ($\sigma_h$) of 68.0 MPa. This result is highlighted as a specific data point in Figure 6-5.

6.4 In Situ Stress Profile

Using the methodology described in Section 6.3, breakouts along the length of the pilot hole were used to evaluate the changes of horizontal stress as a function of depth. It should be noted that 11% of all breakout geometries that were quantified in the borehole could not produce a result within the bounds of the model. These breakout profiles could, therefore, have been influenced by heterogeneities in the rockmass or may also generally demonstrate a drawback of using this method, as not all combinations of breakout shapes are captured throughout the iterative modelling process. From Figure 6-5, the interpreted magnitudes of principal stress appear to be slightly elevated from the regional stress trend (Diederichs, 1999). From stress measurements of nearby mines in the Sudbury basin, the results however do plot within the observed ranges. The summary of Sudbury Basin stress values can be found in Table 6-1.
Table 6-1 – Summary of historical stress values from mines in the Sudbury Basin (Cochrane, 1991).

<table>
<thead>
<tr>
<th></th>
<th>Mining Operation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Otaping</td>
</tr>
<tr>
<td>depth level</td>
<td>4025</td>
</tr>
<tr>
<td>depth (m)</td>
<td>1227.6</td>
</tr>
<tr>
<td>( \sigma_1 )</td>
<td>64.5</td>
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<tr>
<td>plunge</td>
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<tr>
<td>trend</td>
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<tr>
<td>( \sigma_2 )</td>
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<tr>
<td>plunge</td>
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<tr>
<td>trend</td>
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</tr>
<tr>
<td>( K_{xy} )</td>
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</tr>
<tr>
<td>( \sigma_3 )</td>
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</tr>
<tr>
<td>plunge</td>
<td>48.0</td>
</tr>
<tr>
<td>trend</td>
<td>147.0</td>
</tr>
</tbody>
</table>

Source: (Cochrane 1991) 

For the plot of results in Figure 6-5, no water pressure in the borehole at the time of breakout formation was assumed. This was done given the previous indications that at depth in Sudbury, there is typically very little groundwater (Zhang, 2016). Although there is inevitably some water used during the drilling process, the base case interpretation considers the effect of water in the borehole to be negligible. In Section 6.4.2, the impact on stress estimation with consideration for various levels of borehole water is explored.
Figure 6-5 – Principal Horizontal Stress and in-plane stress ratio results from breakout profiles observed in pilot holes FNX-1204 and GT0020VCa. Results are compared to Canadian Shield Stress estimate and stress measurements at several mining operations in the Sudbury Basin (Table 6-1). Red points denote the example measurement previously described in Section 6.3.
6.4.1 Comparison of Pilot Hole Results

When the estimate of stress is reviewed for each borehole, both appear to show relatively similar trends as regional stress measurements. A summary of changes in stress magnitude as a function of depth is as follows:

- Both holes are in close agreement from 900-1000 m, after which the in-plane stress ratios begin to diverge leading to some discrepancy in the estimated magnitudes of stress.
- At 1150 m both holes show a sharp decrease in stress which is associated with somewhat elevated $K_{xy}$ values. This causes the estimated stress to approach the historical average stress from the Canadian Shield (as shown as the black dashed lines).
- From 1350 m onwards, both holes diverge in trend until 1575 m, where both show elevated magnitudes of stress, which is associated with a lower in-plane stress ratio.
- At 1675 m a sharp decrease in stress is seen which is denoted by a modest increase in the $K_{xy}$.
- Following this, both holes return to magnitudes observed prior to the previous drop, until the end of FNX-1204.
- From 1825 m onwards, GT0020VCa intersects frequent high-angle faults which led to very few reliable breakout profiles for the estimate of stress. Due to this, confidence in the stress estimate through this interval is poor.

When compared to the regional trend of stress in the Canadian Shield, both boreholes are generally in agreement with these trends. Although the Major Horizontal Stress in pilot hole GT0020VCa appears to be slightly elevated with respect to the trend line, it does fall within the range of other measurements of stress throughout the Sudbury Basin. In a similar fashion, both the in-plane stress ratios appear to over predict what is typically observed. Despite this, they do appear to be congruent with the other Sudbury Basin stress measurements.

Although both boreholes generally show the same trends in elevated/suppressed stress magnitudes, the deviations in FNX-1204 appear to be more pronounced. This is most apparent at 1100 m
and 1700 m, where it’s profile sharply decreases in estimated stress. Given that the same methodology of evaluating each breakout profile was used and applied to the same numerical modelling database, it may be as a result of the number of breakouts which were quantified in each hole. Although FNX-1204 generally showed less variance in measured opening angle and breakout depth between each breakout profile, it did however have a greater frequency of breakout occurrence. This is shown in Figure 6-6, where a large amount of breakouts were logged in FNX-1204, particularly at the 1600-1800 m interval. If these breakouts produced similar estimates of stress, this could more greatly influence the average stress profile shown in Figure 6-5, thus causing the more prominent deviations in stress estimate.

Figure 6-6 – Opening angle (left) and normalized breakout depth (right) of measured breakouts from ATV survey. Measurement locations are shown as points with the mean breakout angle and breakout depth as the hashed line. An inverse square mean used for trend analysis (solid line).

Another point worth noting is the elevated stress estimate in GT0020VCa at 1600 m and again between 1700-1800 m, that coincides with two intervals of rhyolite. As previously discussed in Chapters 3 and 4, the rhyolite lithology below 1575 m shows a distinct compositional change with a more prominent foliation. The effects of this were observed both in situ during the ATV survey and again during lab testing of various RHY samples in these intervals. Given the abrupt increase in stress through
both intervals, it is possible that using the average rhyolite rock strength acts to over-estimate the strength of the borehole and thus the stress at these locations. It is also conceivable that the chosen model parameters may not fully capture the true failure mechanics in this zone, leading to a discrepancy between what is predicted and what stress is present in situ. The lack of versatility in the modelling process to account for such variations in material behaviour is perhaps the greatest limitation of this method. It is entirely possible that another set of models that better reflect these conditions could be created and used for comparison in these intervals, although this ultimately reduces the efficiency of the proposed method.

6.4.2 Influence of Borehole Fluid Pressure

Given the depth-independent nature of the breakout database, a borehole column pressure ($P_b$) was not explicitly applied during modelling. As a first pass analysis, the wall strength of the rock during the sensitivity analysis was increased by a value equal to the fluid pressure at that point, assuming a certain column height of water was in the hole at the time of breakout. As Walton et al. (2015) discussed, this is the minimum influence that water pressure would have on the borehole. Given that the material becomes increasingly frictional throughout progressive breakout, the fluid pressure may have a greater impact on wall strength than simply adding the existing pressure to the strength term.

Considering the discussion in Section 6.2, the $BWS$ used the crack damage threshold from testing of each lithology. This took the following form:

$$BWS = CD + P_b$$  \hspace{1cm} (6-5)

In the previous analysis of stress, the influence of the presence of borehole fluid on the occurrence of breakout was not considered. This was for two reasons, the first of which is because other investigations have often found the at depth in Sudbury, conditions are relatively dry (Zhang, 2016) and secondly, if water was present at the time of breakout, the actual height of the water column isn’t known.

For the sake of discussion it has been assumed for this derivation of stress that the borehole was filled to surface with water at all times during drilling. When $BWS$ from Eq. (6-5) above, is substituted
into Eq. (6-6), the individual terms defining borehole strength can be divided into two components (Eq. 6-7). From here it is apparent that the estimate of Minor Horizontal Stress must increase by a magnitude equal to the second component in Eq. (6-7). This change in interpreted stress is proportional not only to the water pressure at the point in the borehole, but also the $K_{xy}$ and $R_\sigma$ at that point. From here the Major Horizontal stress is calculated in an identical manner to previously (Eq. 6-8).

$$\sigma_h = \frac{R_\sigma \cdot BWS}{3 \cdot K_{xy} - 1}$$  \hspace{1cm} (6-6)

$$\sigma_h = \frac{R_\sigma \cdot 0.7UCS}{3 \cdot K_{xy} - 1} + \frac{R_\sigma \cdot P_b}{3 \cdot K_{xy} - 1}$$  \hspace{1cm} (6-7)

$$\sigma_H = K_{xy} \cdot \sigma_h$$  \hspace{1cm} (6-8)

When this is applied to the breakout database, the resulting change in estimated stress can be seen in Figure 6-7, where the corrected stress estimates, assuming the borehole is filled to surface, are shown in blue. From this, the percent change in stress was calculated, and is shown on the right of the figure. The increasing change in stress as a function of depth should be anticipated, given that the column of water is constantly increasing as drilling progresses. What should be noted from this is that the change in stress doesn’t perfectly follow the trendline. This is caused by the fact that the increase in stress is not just water pressure dependent, but also reliant on the in-plane stress and tangential wall stress ratios at that location (Eq. 6-7). In cases where the points plot below the trendline, this is a result of an elevated $K_{xy}$ or suppressed $R_\sigma$. If points plot above the average, it is the opposite of the previously described case. Over the length of the hole, the observed change in stress, due to borehole column water, was on the order of 8-19%. This should be regarded as the absolute maximum potential stress in the borehole (assuming the initial strength assumptions are correct). The trendline of stress change is shown in Eq. (6-9), with an average gradient of 0.07% increase per meter of depth. This trend however most likely isn’t fully linear,
particularly in near surface conditions where the confining state is much less. In this situation, in theory, the intercept of this trend should be 0% rather than 1.56%.

\[ \%\sigma_{\text{change}} = 0.07\% \cdot \text{Depth} + 1.56\% \] 

(6-9)

Figure 6-7 – Estimation of Principal Horizontal Stress magnitudes with the consideration of borehole fluid pressure on the borehole strength. Water column is assumed to be at surface for conservative estimate. Change in stress is plotted on the right showing relative increase as a function of depth.

6.4.3 Relationship to Material Properties

6.4.3.1 Material Property Variance

Two options exist when including the potential variance of material properties and the subsequent estimate of stress. The first is taking a reasonable range of material properties to run sets of models that reflect this variability. This would then produce a number of potential stress profiles, which could be averaged to produce a most likely stress scenario. One notable limitation of this is the number of models...
which would be run and interpreted to produce this estimate. Similar to other described techniques, this would be a departure from the intended methodology and efficiency associated with it.

The second, and more viable option, would be to review the stress results based on the statistical distribution of rock strength from laboratory testing. As shown in Figure 6-8, an upper and lower bound of stress was established by plotting the results when the assumed strength is changed by differing amounts. In this case the stress envelope is defined by one standard deviation of strength for each rock type from laboratory testing (Table 6-2).

Some of the largest variation in expected ranges of stress is seen within the quartz diabase intrusions (QDIA), which occur intermittently in small intervals (<30 m) along the length of the hole. This should be expected given the lack of testing that was conducted on this rock type (6 samples), thus leading to a notably higher standard deviation than the other rock types.

Table 6-2 – Summary of mean unconfined compressive strength and standard deviation from laboratory testing. Results are presented and discussed in Chapter 3 and Appendix B.

<table>
<thead>
<tr>
<th>Lithology</th>
<th>CD</th>
<th>Std.dev</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>MPa</td>
<td>MPa</td>
</tr>
<tr>
<td>MTBS</td>
<td>146.3</td>
<td>38.7</td>
</tr>
<tr>
<td>MTGB/MXGB</td>
<td>174.9</td>
<td>21.6</td>
</tr>
<tr>
<td>RHY</td>
<td>170.0</td>
<td>28.7</td>
</tr>
<tr>
<td>QDIA</td>
<td>213.5</td>
<td>45.3</td>
</tr>
</tbody>
</table>
6.4.3.2 Spatial Variation of Strength

When material properties from laboratory testing are compared against the stress profile, a number of trends can be correlated between the two. Figure 6-9 shows the results of peak strength, crack accumulation thresholds, tensile strength and stiffness, compared with the stress profile from pilot hole GT0020VCa. When reviewing these results, it should be kept in mind that the slope of the stress profile should be considered relative to the regional stress trendlines. This means that if the slope of stress estimation is greater than the dashed lines, then this denotes an area with a less-than expected change of stress with depth. This is important when comparing stress with the material properties as it isn’t solely the change in magnitude of stress that should be considered but also how stress is changing relative to what is expected with depth in the Canadian Shield.

Another important concept during this discussion is how a difference in strength between the average lithological value and each point strength value impacts the overall estimation of stress. If at a point along the borehole, the strength of the samples are less than the average strength used to calculate
stress, then this will act to overestimate stress. If the opposite is true, then stress will be underestimated, which is somewhat counterintuitive. With these concepts in mind, the following relationships were observed:

- Elevated stress from 900-1125 m corresponds to slightly elevated lab strength but low Young’s Modulus. This appears to be against what is expected given that elevated stress is often associated with stiffer rock.
- The decrease in trend of stress from 1250-1325 m is accompanied by an associated decrease in stiffness and increase of strength. This is indicative of the situation where the rock strength in this interval is greater than the average strength used to calculate stress, thus produced a suppressed trend in stress.
- The increase in stress from 1325-1475 m is directly proportional to an increase in Young’s Modulus in this interval. From here, interpreted stress briefly decreases until 1525 m, which is reflected in the change of stiffness.
- The interval from 1550-1650 m contains the highly-foliated rhyolite. It is also the location where \(CD\) and \(UCS\) appear to converge, which has been interpreted to be potentially caused by stress induced core damage. From lab testing, the mechanism of failure in the samples also was fully axial and appeared to be strongly influenced by the foliation in the rock. This led to decreased compressive strength and also tensile strength through this interval. Given the potential failure of the model to account for this failure mechanism, and the underestimation of strength relative to the average rhyolite strength, this has led to the pronounced increase in stress.
- From this point onwards, there is no observable trend between intact material properties and estimated stress. In the shear zone, data quality is limited with most breakouts not being quantified due to the influence of structure.
Figure 6-9 – Estimated profile of Principal Horizontal Stresses from pilot hole GT0020VCa, compared to lab testing results of damage thresholds ($CI, CD, UCS$), tensile strength and Young’s Modulus. Details of the lab scale results are discussed in Chapter 3.
6.5 Methodology Limitations

Many of the limitations associated with this work are due to the simplifications required to implement this methodology. This is true of this work and geomechanics design in general. The following is a summary of limitations, as they relate to the estimation of stress:

- Assumed material properties at the borehole scale: From intact lab testing, this provided a variety of results involving the nature of failure (intact/structural). From the inspection of breakout, it is often difficult to resolve what the true mechanism of failure might be. Even if an attempt was made for this, further work on how such structures may impact breakout in a borehole needs to be made, as this varies from the failure of a cylindrical sample of core. Part of the utility of this method is the quickness of evaluating the breakout profiles, rather than taking a specific approach of reviewing each occurrence of failure in detail.

- Inputs to the CWFS constitutive model: Using the results of intact lab testing, it was shown that assuming average material properties had less than a 5% influence on breakout opening angle and depth in the numerical models. Although this supports the proposed methodology, other inputs such as initial/mobilized friction angle, residual cohesion, dilation angle and shear strain increment showed a great influence on breakout formation. It is these parameters that were chosen solely through literature, and ideally should be refined going forwards.

- Variable ground conditions along the borehole: Given the potential length of boreholes where this method could be employed, variation in consistency of rockmass behaviour is highly likely. Although this largely wasn’t observed in these investigated geologies, it is a quite real possibility if various intersected rock types show inherently different failure mechanics. This would then require multiple separate modelling databases to be created to consider this. It is this limitation which could be the most detrimental to the practicality of the outlined method, as it would require a much larger time commitment to resolve the stress state.
• Relationship between rock strength and borehole strength: One of the key assumptions as a model input in Chapter 5 and database assumption in Chapter 6 is the true strength of the borehole wall.

• The interpretation of stress tensor orientation: As shown from other stress measurements in the Sudbury Basin, stress is often at an angle to the vertical direction. This means that in a vertical pilot hole, the assumption that the horizontal stresses are also the Principal Stresses may not always hold true. Despite this, without multiple boreholes at different orientations, this can’t be known.

6.6 Conclusion

Potentially the most important factor when estimating stress through the back analysis of borehole breakout, is a carefully considered effective borehole wall strength. Despite some previous work on this topic by authors such as Martin (1997), much uncertainty remains surrounding the strength of the borehole at the time of breakout. Considering that the crack damage threshold is the upper-bound of long term in situ strength, it was decided to adopt an effective strength reflecting this. This produced a stress estimate from both pilot holes that closely relates to the trends of stress in the Canadian Shield and point measurements of stress from other mines surrounding the Sudbury Basin.

The consideration of the influence of borehole fluid pressure on the occurrence of breakout and subsequent prediction of stress was made. This included reviewing the effects on the Principal Stress magnitudes as if the borehole was assumed to be filled to surface with water at all times during drilling. This created a relative increase in the Major Principal Stress of 8-19%, increasing with depth. The variations of material properties as a function of depth were also compared to assess the influence which such parameters may have on the final estimation of stress. Finally, certain limitations and considerations of the proposed framework for estimating stress were explored, with emphasis on how it may be applied across a broad range of geological conditions.
Chapter 7

Methodology Application

7.1 Introduction
This chapter provides a discussion on various implementation strategies of the proposed stress technique and further academic work that can be done to verify the results. This takes a specific look at how it could apply to the results at the Victoria Project and also more broadly in the current framework of stress estimation. The following concepts are explored throughout:

- Combining stress results from multiple boreholes to create a statistical distribution of stress with depth.
- Using a database of boreholes and ATV surveys to create a three-dimensional stress model of an entire project.
- Upscaling of breakout models for use in prediction of excavation overbreak.
- The further academic development of the method through a more detailed sensitivity analysis of material properties and use of horizontal boreholes to validate the current stress estimates from the two pilot holes.

7.2 Stress Result Integration
If the proposed methodology is implemented over the time-span of an entire project, this can allow for a detailed integration of results across multiple boreholes and other stress estimation methods. With the continuous profile of stress that can be deduced from borehole breakout, this provides a unique opportunity to increase confidence in estimated stress by combining results or creating a 3D stress model of the project through interpolation of individual profiles. Although this is beginning to become prevalent in the oil and gas industry through other methods of stress determination, no real attempt has been made to spatially quantify stress in the mining industry. This can be attributed to several factors that include:

- Cost of currently accepted methods.
• Each measurement representing a point estimate of stress.

• Uncertainty associated with measurement results.

• Operational limitations, including short-dry holes for overcoring or a non-reverse faulting regime for hydraulic fracturing (refer to Chapter 2).

With the use of the proposed methodology of continuously assessing breakout along the length of a borehole, this sort of data resolution can be used to overcome the obstacles currently facing the mining industry with respect to creation of a comprehensive stress model. This can be through the statistical approach of combining borehole results to define a representative “average” stress tensor or the creation of a three-dimensional stress model, which both represent notable improvement over current practices.

7.2.1 Result Amalgamation

The first option once several boreholes are surveyed for breakout, is to combine the results from each into an estimate of stress. Ideally these boreholes would be of the same orientation and relatively closely spaced. If this is not the case, the lateral variability in stress state would need to be considered. If borehole spacing is relatively large, interpretation of the differences in stress would need to occur to determine if it is reflective of actual spatial variability, in which case, a 3D stress model may be more appropriate.

The results from the boreholes can be amalgamated to create a “most likely” stress estimation with reasonable upper and lower bounds. This is shown in Figure 7-1, using four different fabricated borehole breakout profiles, which are then averaged to produce the final estimate, with a ±1 standard deviation window of stress magnitudes. The assumption in this case is that breakout azimuths between each hole are the same and stress can be added with corrections for orientation. If this is not the case, then the tensor must be manipulated prior to any averaging, as discussed in the following section.
Figure 7-1 – Theoretical analysis of estimates of stress from a) multiple boreholes with b) the average stress profile from the boreholes with envelope as defined by +/- 1 standard deviation.

As seen in Figure 7-1b, the upper and lower bounds act as a confidence interval, where, when the envelope is close to the mean stress line, this denotes that the boreholes generally all predicted similar stress magnitudes. This is evident in the first 200 m of the plot where the range of predicted stress is relatively small with it gradually diverging towards the bottom, as each borehole predicts a more variable stress state.
7.2.1.1 Tensor Manipulation

As previously discussed, stress is a tensor quantity and any manipulation of results must reflect this fact. It should be kept in mind when any sort of post-processing result amalgamation is proposed. When several stress tensors are compared, it does not suffice to determine the mean stress state between them by simply taking an average of their magnitudes and orientations. Each tensor must first be divided into its respective components, where the average of each component can then be taken and turned back into principal stress space. This appears as follows:

\[
\text{Mean} = \left\{ \begin{array}{ccc}
\sigma_{xx}^A & \tau_{xy}^A & \tau_{xz}^A \\
\tau_{yx}^A & \sigma_{yy}^A & \tau_{yz}^A \\
\tau_{zx}^A & \tau_{zy}^A & \sigma_{zz}^A
\end{array} \right\} + \left\{ \begin{array}{ccc}
\sigma_{xx}^B & \tau_{xy}^B & \tau_{xz}^B \\
\tau_{yx}^B & \sigma_{yy}^B & \tau_{yz}^B \\
\tau_{zx}^B & \tau_{zy}^B & \sigma_{zz}^B
\end{array} \right\} / 2
\]

\[
= \left\{ \begin{array}{ccc}
\sigma_{xx}^{(A+B)} & \tau_{xy}^{(A+B)} & \tau_{xz}^{(A+B)} \\
\tau_{yx}^{(A+B)} & \sigma_{yy}^{(A+B)} & \tau_{yz}^{(A+B)} \\
\tau_{zx}^{(A+B)} & \tau_{zy}^{(A+B)} & \sigma_{zz}^{(A+B)}
\end{array} \right\} / 2
\]

where \( A \) and \( B \) are the tensors being averaged to form each component, \( \sigma_{ii}^{(A+B)} \) and \( \tau_{ij}^{(A+B)} \), of the resulting matrix.

7.2.2 3D Stress Model Generation

Given the ease of completing ATV surveys on existing boreholes, as a project develops, and a large quantity of data can be collected, this could facilitate the creation of a 3D stress model. This is particularly true in cases where development of a deposit begins and infill drilling can be used to assess breakout in boreholes at various orientations, to validate the earlier results from surface drilling. During the NI 43-101 reporting procedure, this dictates that a measured mineral resource must be delineated by holes that are generally at less than 100 m spacing. From this, a strong spatial volume of breakout measurements can be taken across the span of an entire deposit.

From the results of stress in each borehole, an interpolation technique such as inverse distance weighted mean or the kriging approach can be taken to infer the spatial changes of stress between
boreholes. In this case, no manipulation of the stress tensor needs to be done, as the orientation of stress is also a direct input to the 3D model. From the review of literature, no fully-3D model of stress has been created for use in the mining industry, although some attempts have been made in the oil and gas industry. An example of this is shown in Figure 7-2, where the changes of effective stress around a salt dome is shown using the results from a tomographic seismic survey coupled with downhole pore-pressure results (den Boer et al., 2011). Although the technique for estimating stress varies from what was used in this model, the same sort of stress model can be created and used as an input to the life-of-mine model or during more specific modelling of mine development as an operation matures.

Figure 7-2 – 3D effective stress model of salt dome in the Gulf of Mexico, using downhole geophysical surveys along boreholes, shown as the white lines (den Boer et al., 2011).

The 3D model can be continuously refined based on the addition of other stress results, or indicators of stress such as excavation damage mapping or mine induced seismicity. In the cases where
the predicted stress model has difficulty reconciling what is observed from these indicators, a back analysis of stress can be completed to assess either what stress would be required to cause such damage, or conversely what material properties would be needed to match damage with the predicted stress.

7.3 Upscaling to Excavation Overbreak

From the profile of stress in pilot hole GT0020VCa in Chapter 6, the magnitude of potential shaft overbreak was evaluated based on the size of each EDZ from the curves shown in Chapter 5. The extent of predicted damage was kept as a function of depth normalized to the shaft radius, as both the production and ventilation shafts differ in planned diameters, being 7.6 m and 6.8 m respectively. Overbreak was characterized based on a range of potential excavation strengths, with the practical implementation of such an approach demonstrated in this chapter.

7.3.1 Excavation Strength

Similar to the discussions surrounding the strength of the borehole wall for the prediction of stress, the same consideration must be made when determining excavation overbreak. At this scale, it is not only grain-scale heterogeneity such as foliation and healed structure which influences strength, but also the presence of discontinuities such as joints and faults. As the creation of the modelling database considered the failure surrounding the excavation as fully intact, this assumes that structure at the excavation scale doesn’t play a significant role on defining the strength of the excavation. Hoek et al. (2002) proposed a set of criteria to define the transition between a more conventional shear-dominated failure along discontinuities and intact failure, known as spalling. This criteria considers the rockmass $GSI$, the intact UCS and the tensile strength. Figure 7-3 shows the values of $GSI$ along the length of the borehole, from 900 m to the end of the hole. Overbreak wasn’t considered above this point as no estimates of stress exist from the back analysis of borehole breakout. $GSI$ was calculated based on the empirical relationship with $RMR_{1989}$ by Bieniawski (1989); as follows:

$$GSI = RMR_{1989} - 5$$

(7-2)
Figure 7-3 – Summary of GSI from pilot hole GT0020VCa, based on the relationship presented in Eq. (7-2).

From the plot of GSI, it can be seen that on average, the range is between 65 and 75. Some zones of decreased rockmass competency exist between 900-1000 m and surrounding the shear zones at 1825 m. The other important metric in determining the probable mechanics of failure include the ratio of intact strength to tensile strength. A summary of results from laboratory testing is shown in Table 7-1, where the ratio ranges from 16.2 in the metabasalt to 22.7 in the metagabbro. With both the GSI and strength ratio considered, Table 7-2 from Hoek et al. (2002), shows that most failure along the shaft will be governed by intact mechanics, although areas of lesser competency may experience a mixed mode failure with some component of shear. At these locations of decreased GSI, caution should be taken when reviewing the following estimation of overbreak.
Table 7-1 – Summary of peak strength and tensile strength from laboratory testing of samples from pilot hole GT0020VCa. Full description can be found in Chapter 3/Appendix B.

<table>
<thead>
<tr>
<th>Lithology</th>
<th>UCS</th>
<th>TS</th>
<th>UCS/TS</th>
</tr>
</thead>
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<tr>
<td>MTBS</td>
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<td>16.2</td>
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<td>MTGB/MXGB</td>
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<td>RHY</td>
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<tr>
<td>QDIA</td>
<td>297.1</td>
<td>14.6</td>
<td>20.4</td>
</tr>
</tbody>
</table>

Table 7-2 – Probable mechanism of failure based on results from Figure 7-3 and Table 7-1. GSI denotes a conventional shear based failure while SP denotes spalling (after Hoek et al. 2002).

Numerous authors have shown that failure in massive hard rock excavations occur when the tangential stress exceeds between 33%-50% of the rock’s intact strength (Pelli, P.K. Kaiser and Morgenstein, 1991; Martin, Kaiser and McCreath, 1999). This was established by the back-analysis of displacement surrounding various excavations in massive hard rock, such as at the URL. This lower bound of strength correlates well in these cases with the crack initiation threshold from laboratory testing. Given that the results from the intact testing of samples from GT0020VCa have an average crack initiation of 41%UCS, this was selected as the lower bound of in situ excavation strength for the analysis. As previously discussed in Chapter 6, the upper bound of material strength at any scale is considered it’s crack damage threshold (Bieniawski, 1967; Martin, 1997; Diederichs, 2007; Cai, 2010). The maximum excavation strength was therefore considered to be the same as the borehole strength of CD.
7.4 Overbreak Characterization

Similar to the methodology for stress determination, estimating shaft overbreak could be done either numerically or through the graphical approach. In the case of the numerical solution, no re-arrangement of Eqs. (7-3) and (7-4) would need to be done to arrive at the estimation of overbreak. By choosing the stress state at every 10 m depth from the stress profile, these equations could simply be solved, considering the excavation wall strength and appropriate fit constants (Table 7-1), to estimate depth and extent of overbreak.

\[
BOA = e^{(a_1 + b_1 \frac{\ln(K_{xy})}{K_{xy}} + \frac{c_1}{\ln(R_\sigma - d_1) + \frac{e_1}{R_\sigma - d_1}})}
\]

(7-3)

\[
NBD = \frac{a_2 + b_2 \cdot \ln(K_{xy}) + c_2 \cdot \ln(R_\sigma - d_2)}{1 + e_2 \cdot \ln(K_{xy}) + f_2 \cdot \ln(K_{xy})^2 + g_2 \cdot \ln(R_\sigma - d_2)}
\]

(7-4)

Table 7-3 – Fit constants used in Eqs. 7-3 & 7-4, to develop the graphical representation in Figure 7-4 and Figure 7-5.

<table>
<thead>
<tr>
<th></th>
<th>HDZ</th>
<th>HDZ</th>
<th>EDZi</th>
<th>EDZi</th>
<th>EDZo</th>
<th>EDZo</th>
</tr>
</thead>
<tbody>
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<td>(i=2)</td>
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<td>0.10</td>
<td>0.10</td>
<td>0.10</td>
<td>0.10</td>
<td>0.05</td>
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</tr>
<tr>
<td>(e_i)</td>
<td>-4.67</td>
<td>0.17</td>
<td>-4.04</td>
<td>1.37</td>
<td>-3.54</td>
<td>0.72</td>
</tr>
<tr>
<td>(f_i)</td>
<td>-</td>
<td>-0.03</td>
<td>-</td>
<td>-0.03</td>
<td>-</td>
<td>-0.05</td>
</tr>
<tr>
<td>(g_i)</td>
<td>-</td>
<td>-0.88</td>
<td>-</td>
<td>-0.79</td>
<td>-</td>
<td>-0.14</td>
</tr>
</tbody>
</table>

If however the graphical approach is more desirable, the opposite process from the one described when estimating stress could be used. This would follow:

1) At 10 m intervals determine the magnitude of principal horizontal stresses and in-plane stress ratio, \(K_{xy}\).

2) For the given estimate of excavation wall strength, determine the appropriate \(R_\sigma\).

3) From the BOA and NBD plots for each corresponding EDZ profile locate the curve and x-axis location corresponding to the stress ratios.
4) Connect a vertical line between these points, then draw a horizontal line to the y-axis to determine the respective overbreak depth and opening angle.

The same shortcomings of using the graphical approach for estimating stress are prevalent in this method, as each plot doesn’t show every possible combination of in-plane stress magnitude. This therefore leads to some estimating, when the $K_{xy}$ isn’t exactly as shown on the plot.

For the example shown in Figure 7-4, this is considering a wall strength of 50% $UCS$ to represent the relationship between each of the damage profiles. Figure 7-5 shows how the assumed excavation scale influences the overall estimate of excavation damage. The results of this are further explored in the following section.
Figure 7-4 – Prediction of EDZ sizes along the length of the proposed KGHM Victoria shafts. Each respective zone was determined using the predicted stress from Chapter 6 and an estimated excavation strength of 50% UCS as an input to Eqs. (7-3) & (7-4) with the appropriate fit constants for each EDZ type.
Figure 7-5 – Results of considering various excavation strengths on the prediction of shaft overbreak geometry (opening angle and depth). Ranges of strength vary from crack initiation (CI) to crack damage (CD).
7.4.1 Result Interpretation

As can be seen in Figure 7-5, the results of predicted overbreak magnitudes appear to somewhat mirror the deviations in predicted stress along the length of the hole. At the top of the interval in question, both the predicted opening angle and depth for each respective damage zone shows an elevated predicted amount. This decreases from 1125 m to 1575 m, at which point a sharp increase is observed, corresponding to the highly-foliated rhyolite unit. In this case, this is further indication that the strength of rhyolite is an overestimate in this particular zone, which is supported by the elevated predicted stress and the observed intact failure mechanics of samples in this interval.

When each damage zone (HDZ, EDZi and EDZo) is compared, they show the same trends. The outer EDZ profile shows the most sensitivity to stress change, which from the modelling results, can be seen in the shape of the curve fits. For the EDZo, the curve fits show the steepest gradients and therefore the most sensitivity to changes in stress on overall magnitude of overbreak. As previously discussed in Chapter 5, the opening angle of HDZ and EDZi are almost identical along the length of the hole. This further supports the previous statement that, these two damage extents could be treated as the same.

When the assumed excavation strength is changed from \( CI \) to \( CD \), the EDZo shows the greatest change in predicted extent. Similar to the previous discussion of the sensitivity of this zone to stress and inherently strength change, this should come as no surprise. The highly damaged zone showed the least sensitivity to strength change, which is noteworthy given its importance to the short-term stability of the shaft. During development, these predicted ranges of overbreak can be continuously reviewed against observed shaft damage, to assess what the true effective strength may be. This is all done with the assumption that structure frequency over the majority of the shaft has little influence on the overall excavation behaviour. Given that the average \( GSI \) of the rockmass is 69 along the length of the pilot hole, this should hold through most portions of construction (Hoek et al. 2002).
7.4.2 Overbreak Discussion

In the context of shaft design, the HDZ envelope may be considered to be the depth of immediate failure post-excitation, prior to the installation of support. If scaling work is done preceding the implementation of permanent support and/or a liner, the EDZi limit then becomes the absolute maximum potential extent of overbreak. Although traditionally the EDZo is more of a concern for longer-term infrastructure (such as nuclear waste repositories), this boundary could act as a proxy for predicting locations where increased long-term deformation can occur. This is due to the micro-damage in this zone increasing the potential for long term creep, strength degradation and in some rock types swelling due to increased fluid or atmospheric intrusion into the damaged rock (Steiner, Kaiser and Spaun, 2011). Delayed deformations, could lead to an increased wear on the shaft infrastructure from misalignment of track guides, particularly if there is groundwater within the micro-cracks throughout the EDZo. Although this may not be of a concern in crystalline rock such as what is seen at the Victoria Project, in other highly-weathered or sedimentary geologies this may be an issue during the mine’s life.

If the assumption that the same mechanics which govern borehole-scale breakout are also the predominant drivers of failure at the excavation-scale, this can be used for the prediction of excavation overbreak. This is particularly true when the rockmass under consideration is deemed to be relatively free of systematic structure. As is shown in Figure 7-6, the presence of jointing can greatly influence the symmetry and size of excavation failure. In the case of a highly anisotropic medium, caution must be taken when deciding on the effective strength of the excavation and whether the assumption that it fails according to brittle mechanics remains correct at this scale. In highly jointed ground, a failure criterion which is initially governed by friction would be more appropriate than the CWFS model proposed for brittle failure. The approach taken of backing out overbreak from an assumed wall strength is closely based on modelling and previous literature from work on circular openings. Due to this, applying this methodology to non-circular openings such as mine drifts and irregularly shaped stopes, is not advisable.
Alternatively, a more explicit approach of modelling excavation overbreak can be taken, using the material properties from lab testing and joint orientation/frequency from the ATV survey. This approach would allow for the consideration of influence of structures on the overall shape of overbreak, as shown in Figure 7-6. This however does still require some assumptions of what the intact strength of the rock is, similar to what is shown when using the generalized curve approach. This also requires some assumption of the joint strength, which could be derived from core logging results of joint condition, roughness and alteration.

Given the computational intensity of a discontinuum model, a three-dimensional analysis would have to be done at a discrete interval to practically implement explicit discontinuities. This then can only assess overbreak over a limited range, unless multiple models are run. Another potential limitation to this approach is the fact that the mapping of borehole scale fractures provides no insight into the persistence of a joint. This is illustrated in Figure 7-7, using the joint set orientations at 1550 m depth in rhyolite of pilot hole GT0020VCa. In this case, the foliation is assumed to form an infinitely persistent joint set while the other two are given an arbitrary mean joint length that varies from 0.5 m to 3.0 m. Across each model,
the sum of joint lengths are kept constant, such that the model with the shorter mean lengths has the greatest number of joints.

From Figure 7-7, it is conceivable that the results of shaft overbreak in each of these modelling cases could be quite different. Although this approach will certainly provide a more detailed result of excavation damage than the generalized approach, it too is limited to the scale of data that can be collected during a borehole investigation.

Figure 7-7 – Set of discrete fracture network models created considering the 7.5 m diameter Victoria production shaft and jointing conditions at 1550 m depth in rhyolite, as logged by ATV survey. Models vary in mean joint length from; a) 0.5 m, b) 1.0 m, c) 2.0 m and, d) 3.0 m.

7.5 Forward Thinking
From the results presented in this thesis, there are a number of directions that this research can conceivably take to further the confidence and applicability of this methodology. This can include a more
thorough sensitivity analysis of material parameters on the actual prediction of stress, or the incorporation of multiple boreholes at different orientations to verify the results from the vertically oriented pilot holes.

7.5.1 Detailed Review of Material Property Assumptions

As a part of validating modelling assumptions in Chapter 5, a sensitivity analysis was completed considering each material input’s effect on breakout shape. This can act as a proxy of how the estimation of stress may change as a result of the input, however with the complexity of the curve fit solutions, the actual magnitude of stress change can’t directly be determined using this approach. For a more detailed analysis, an entire set of models can be run again with the newly assumed material property, to create a new curve-fit solution. An example of this is shown in Figure 7-8, where dilation angle was modelled at 0° and the resulting curve fits were compared against the curves from the base case in Chapter 5. This shows how breakout shape changes as a function of in-plane stress and tangential wall stress ratios. As can be seen from this case, a $K_{xy}$ closer to unity was less sensitive to changes in dilation angle, although was more sensitive than the other in-plane stress ratios with a change in tangential wall stress ratio.

![Graph](image)

**Figure 7-8 – % change of breakout opening angle (left) and normalized depth (right) as a function of in-plane stress and tangential wall stress ratio. Results consider a model of 0° dilation compared to the base model case.**

From the curve fits of the base case and the proposed dilation model, a set of arbitrary breakout shapes can be solved in each, either numerically or graphically. The resulting estimate of stress can be compared between the two models and averaged across the considered breakout geometries. In this case,
the results of the analysis are given in Table 7-4, where the average in-plane stress ratio is under-
estimated and tangential wall stress ratio is over-estimated with respect to the base model inputs. This
translates into an increased estimate of both Principal Stresses, with the Minor Stress showing the greatest
change.

Table 7-4 – Average change in stress between a dilation= 0° curve-fit to the base case curves used to
estimate stress in Chapter 6.

<table>
<thead>
<tr>
<th></th>
<th>Average % Change</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$K_{xy}$</td>
</tr>
<tr>
<td>$\Psi= 0^\circ$</td>
<td>-9%</td>
</tr>
</tbody>
</table>

If this process is done for each end member of potential material property inputs, this can provide
a greater insight into how stress estimation using borehole breakout can change from each input. This will
further narrow down which material assumptions need to be more carefully considered when building the
numerical model database.

7.5.2 Horizontal Borehole Consideration

Once underground development begins, horizontal boreholes can be drilled away from excavations to
review breakout occurrence. A distinct advantage of this is that one of the in-plane components of stress
can be estimated based on the weight of overburden. This simplifies the solution such that only one
component of stress needs to be solved for. This is ideally done once the orientation of the Principal
Horizontal Stress components is known so that the horizontal boreholes are drilled orthogonal to these. If
this isn’t the case and the borehole is at some angle to the Principal Stresses then the resulting stress
prediction would be some component of stress, which could only be resolved if multiple horizontal
boreholes at different angles were used.

An ideal setup of using horizontal boreholes would involve the combination of two horizontal
holes with a vertical hole, as shown in Figure 7-9. In this case the results of stress magnitude and
orientation from the vertical hole will be reviewed first. Upon confirming the orientation of stress, the
horizontal holes can be drilled parallel to both horizontal components of stress. The estimates of stress magnitudes from all three boreholes can then be compared, with emphasis on how the horizontal stress from the vertical borehole compares to the results from the horizontal holes. This can be most easily done by comparing the $K_{xy}$ ratio from the vertical borehole with the same ratio taken from the horizontal component of stress from each of the other two holes.

![Diagram of breakout results at different orientations with respect to the Principal Stress axis.](image)

**Figure 7-9** – The use of horizontal and vertical boreholes to acquire breakout results at different orientations with respect to the Principal Stress axis.

This process would naturally bring in the discussion of the influence of rock anisotropy on breakout and how important it is to consider this in the modelling stages. Given the complexity of the data acquisition and interpretation using several differently oriented boreholes, this could be valuable as an academic exercise to provide further insight into the validity of the material generalizations and how closely the predicted results from breakout match other measuring techniques.

One of the benefits in a mine is that horizontal holes can be used at each mining level to create a similar continuity in stress estimation that a vertical borehole would give. The results at each level could
however be compared to the vertical stress magnitudes determined based on the weight of overburden. A calibration of the results could then be applied into the overall stress estimation framework.

7.6 Conclusion

The discussions of this chapter provide a number of directions which the approach of stress estimation through borehole breakout can take. This is specific to the Victoria Deposit, through the prediction of overbreak using the generalized database approach or a more explicit modelling of the rockmass. The discussion also involves the consideration of using multiple boreholes across an entire deposit to either develop an average stress state with depth or a more comprehensive three-dimensional model. It is in this latter case where important parameters such as excavation overbreak or ore dilution can be more accurately assessed spatially throughout the mine. This could aid in the selection of more appropriate development sequencing at earlier stages in a mining project.

To further increase confidence in the proposed methodology, several further academic exercises were proposed which include a detailed assessment of the sensitivity of stress prediction on various material inputs, and the use of horizontal boreholes in conjunction with the pilot holes to verify the original stress estimates. Although they may not fit within the realm of practical application, they are however important to consider if this methodology is to be brought into the currently accepted framework of stress estimation.
Chapter 8

Summary and Conclusions

8.1 Summary of Findings

The complete body of findings described in this dissertation can be found in Chapters 3 to 7. This is in such a way to allow the reader to follow the logical work-flow that is proposed to use this technique in practice, with some additional investigation into practical applications in Chapter 7. This took the following sequence:

- Chapter 3- Description of in situ ground conditions through; core logging, downhole ATV surveys and intact lab testing which was completed as part of this thesis. Results, where applicable, were compared between the two existing pilot holes.

- Chapter 4- Review of processing techniques used to quantify breakout in both pilot holes. Known techniques of using breakout to assess the orientation of principal stress were employed, with a brief discussion on using core disking geometry in the oriented portion of pilot hole GT0020VCa to verify in-plane principal stress directions.

- Chapter 5- The use of continuum based modelling for the creation of a generalized database of models to be used to back analyze breakout for the prediction of stress and overbreak.

- Chapter 6- The predicted stress state in both pilot holes are outlined using the generalized database established in Chapter 5. Considerations of the proposed methodology are explored, which include; the effect of borehole fluid pressure on breakout, the relationship of material properties to predicted stress and current limitations of the approach.

- Chapter 7- A set of ways to practically implement this methodology within the stress estimation framework is proposed. This includes how the same approach can be up-scaled to predict excavation overbreak, how multiple boreholes can be used to increase the spatial understanding of stress, and future considerations to improve confidence in the proposed methodology.
8.1.1 Characterization of Rockmass Conditions

As part of this dissertation, two vertical boreholes were characterized based on the geomechanical conditions. The following is a summary of what was considered and compared for each hole:

- Lithology description of each rock type
- Comparison of rockmass condition based on RMR and $Q'$
- Interpretation of joint set orientations from acoustic televiewer survey for each lithology
- Description of intact lab testing results from borehole FNX-1204
- Methodology of sample collection, preparation and testing for intact samples from borehole GT0020VCa. Brazilian tensile (41 samples) and unconfined compressive strength (47 samples) tests were evaluated on a statistical basis considering the effect of lithology on strength and failure mechanism.

From the review of core during the logging process, the differences in mineralogy, grain size and structural fabric were noted. Of the rock types intersected along the hole, the most heterogeneous was the rhyolite. This unit contained quartz amygdules which varied in intensity based on depth. These features appeared to be preferentially oriented along the foliation in the rock. The rock fabric occurred in differing intensity, with the most intense foliation occurring intermittently between 1575 m and 1800 mbs. Of the other units, metagrabbro/metacrystic gabbro showed a large variation in grain size, with the lower end-member being aphanitic while the coarsest samples had grains occurring at the 5 mm scale. From sample testing, it was seen that the coarser grained samples had, on average, the highest strength. Despite the attempt to separate the two end members into distinct units, it was difficult given the gradational change in grain size that was seen in this lithology.

From previous core logging on each hole the rockmass conditions were reviewed. Data reliability issues exist in the former FNX-1204 hole regarding the joint condition and as such data from that hole was largely disregarded. From GT0020VCa, the Norwegian Q system would vary from; Fair ($Q' = 4-10$) to Good ($Q' = 10-40$) quality. From the RMR system, the rock grades at; Good (60-80) to Very Good (80-
This was seen to be relatively constant along the length of the hole, with the exclusion of a set of fault zones near surface and near the end of the hole beginning at 1825 m, where a steeply dipping shear zone was intersected.

Lab testing of samples collected from pilot hole GT0020VCa was completed at the Queen’s University Geomechanics Testing Lab. Samples had an average peak strength that ranged from 196 MPa in the rhyolite to 251 MPa in the quartz diabase intrusions. Brazilian strength also ranged from 12.7 MPa in the rhyolite to 14.6 MPa in the quartz diabase. During lab testing, if failure was either fully through intact material or partially along healed structure, *UCS* samples failed axially and with a high amount of energy. This is with the exception of several samples of rhyolite in the highly-foliated interval, that failed along the rock fabric.

### 8.1.2 Quantification of Borehole Breakout

Collection of breakout data was done using a downhole ATV survey to record shape and orientation of breakout. A criterion for selecting breakout profiles which are indicative of the in situ stress state was developed. This includes selecting breakout which doesn’t appear to be influenced by discontinuities, lithological contacts, quartz veining and shows an enlargement of borehole radius by a magnitude greater than the average rock grain size. Breakouts which met these stipulations were then characterized based on the opening angle and depth of breakout. For pilot holes FNX-1204 and GT0020VCa, 255 and 212 profiles met the criteria and were reviewed for the prediction of stress in Chapter 6.

From the review of breakout orientation, it was established that the average Major Horizontal Stress is oriented approximately East/West. Through the logging of stress induced disking geometry within the oriented portion of GT0020VCa, this method confirmed the results of stress orientation from the occurrence of breakout.

In the highly-foliated interval of rhyolite, a small deviation in stress orientation was observed. The potential effects of foliation on breakout shape and occurrence was investigated to demonstrate the relationship between foliation orientation and breakout geometry. From this investigation, it was seen that
breakout tends to preferentially align parallel to the foliation’s dip direction. At higher in-plane stress ratios this appears to be less prominent. The breakout opening angle also was influenced by foliation, where at lower $K_{xy}$, the opening angle saw a notable increase. To fully establish the influence of foliation, a greater amount of lab testing would be needed.

8.1.3 Technique for Estimating Stress and Overbreak

For the estimation of in situ stress from the breakout profiles that were identified in Chapter 4, a set of models were created to back analyze stress. In the database of models, 180 individual iterations at varying states of stress were created. Between each subsequent run, the in-plane stress and tangential wall stress ratio were incrementally changed to compare against the in situ breakout. For this reason, and to maintain the efficiency of the proposed methodology, the average material properties across the lab testing suite were used. To account for this, and minimize the associated error, the tangential wall stress was normalized to the strength of the borehole. In theory, this allows for the different lithologies to be assessed in parallel regardless of the strength discrepancies.

A sensitivity analysis of the model inputs was completed considering how changes in such properties influenced the shape of breakout. In the case of material properties from lab testing that were averaged for model input, they were reviewed to determine whether this process had the potential to influence the final result of stress prediction. This includes the crack damage threshold, used in the calculation of tangential wall stress ratio, the tensile strength, and stiffness. In the case of CD, this was of importance to validate the assumption that despite the lithologies having a significant variance in strength, they could be reviewed in parallel given the stress normalization in the model. For all lab derived values, the sensitivity of the extent of breakout was less than 5% for both depth and opening angle. Other material properties such as initial/mobilized friction, dilation, residual cohesion and plastic strain increment were also considered. These values were all determined based on reported literature in similar rock types. It was found that these properties showed a much greater influence on breakout shape with
most showing a minimum of 10% increase across the studied ranges. This supports a careful consideration of these values based on published literature when establishing the material inputs.

From the sets of curves, the in situ breakout profiles from each borehole were compared to assess stress in each case. This produced an estimated in-plane horizontal stress ratio and subsequent magnitudes of Principal Horizontal Stresses. This required an assumption of the effective borehole wall strength. From the work of several authors, it was decided to set the strength to the crack damage threshold (from lab testing), which has been shown to be the upper bound in situ strength of an excavation rock (Bieniawski, 1967; Martin, 1997; Diederichs, 2007; Cai, 2010). From this, each breakout profile produced an estimate of stress that was fit with a moving average to create a profile of stress as a function of depth in each pilot hole. Upon inspection of the stress profiles, each borehole is in relative agreement with the average stress trend in the Canadian Shield and with regional measurements taken at other mines in the Sudbury Basin.

A number of assumptions made throughout the process of evaluating stress and overbreak were explored as part of this dissertation. The first includes the potential effects of borehole water on the predicted stress magnitudes. As the creation of the modelling database was depth-independent, a borehole pressure could not be explicitly modelled. Rather, it was considered by increasing the effective borehole wall strength by a magnitude equal to the assumed borehole pressure at that location. As part of this, it was assumed that at all times during progression of borehole GT0020VCa, the borehole was filled to surface. When stress was re-calculated using the new \( BWS \) this produced a change in stress gradient as follows:

\[
\% \sigma_{\text{change}} = 0.07 \% \cdot Depth + 1.56% \tag{8-1}
\]

It should be noted that at each location, the change in stress didn’t fall perfectly on this trend. This is due to the inherent relationship between in-plane stress and tangential wall stress ratios on the effect of borehole water pressure. Due to this, if \( K_{xy} \) was above or \( R_\sigma \) was below average, this produced a change in stress from borehole fluid above the shown trendline. If the material property assumptions that
were made are assumed to be true, then this analysis of the influence of borehole fluid pressure should represent the absolute maximum potential stress profile along the borehole.

### 8.1.4 Proposed Practical Implementation of Methodology

The work of Chapter 7 was aimed at addressing a number of directions which the approach of stress estimation from borehole breakout can take. This is specific to the Victoria Deposit, through the prediction of overbreak using the generalized database approach or a more explicit modelling of the rockmass. The discussion also involves the consideration of using multiple boreholes across an entire deposit to either develop an average stress state with depth or a more comprehensive three-dimensional model. It is in this latter case where important parameters such as excavation overbreak or ore dilution can be more accurately assessed spatially throughout the mine. This could aid in the selection of more appropriate development sequencing at earlier stages in a mining project.

For the quantification of overbreak, this was done using the concept of excavation damage zonation which considers three distinct zones. These include the highly-damaged zone (HDZ) which is denoted by the interconnection of macro-structures with very little remaining confining stress. More distal from the HDZ is the transition to the EDZ which is sub-divided into EDZ$_i$ and EDZ$_o$, with the inner having remaining interconnection of induced structure with only isolated damage towards the outer boundary (EDZ$_o$). Similar to the estimation of stress, the prediction overbreak requires some knowledge of the effective wall strength. The first consideration was what effect discontinuities would have on failure behaviour. Given that most intervals had a recorded $GSI$ of greater than 65, Hoek (2002) showed that this range of rockmass competency would show primarily spalling behaviour. This allowed the same CWFS modelling approach to be used when determining damage zonation. The same curve-fit approach was used, where the stress estimates at 10 m intervals in depth were used to then predict the opening angle and depth of overbreak for each zone.

The results of the overbreak analysis shows elevated levels of overbreak between 900-1150 m, with a sharp increase in overbreak between 1575 m and 1700 m. This occurs within the highly-foliated
rhyolite which is further indication that the lab-scale strength of rhyolite is an overestimate in this particular zone. This is also supported by the elevated predicted stress and the observed intact failure mechanics of samples in this interval.

To further increase confidence in the proposed methodology, several further academic exercises were proposed which included a detailed assessment of the sensitivity of stress prediction on various material inputs, and the use of horizontal boreholes in conjunction with the pilot holes to verify the original stress estimates. In the case of material property sensitivity, a set of models was run considering a dilation angle of 0°, to create a set of unique curve fits. This allowed for the quantification of change in breakout shape not just at one stress (as done in Chapter 5) but across the entire range of $K_{xy}$ and $R_{n}$. In this case, breakout was less sensitive to dilation at lower in-plane stress ratios. Sets of breakout shapes were also numerically solved for using the equations from the base case and the newly derived equations using the dilation=0° model. This showed that the average in-plane stress ratio is under-estimated and tangential wall stress ratio is over-estimated with respect to the base model inputs. This translates into an increased estimate of both in-plane Principal Stresses, with the Minor Stress showing the greatest change.

Although this sensitivity approach doesn’t fit within the realm of practical application, it is important to consider if this methodology is to be brought into the currently accepted framework of stress estimation.

8.2 Research Considerations

Throughout this body of research, there has been much discussion on the necessary considerations which must be made at each step in the process, and also the associated shortcomings of the proposed methodology. Like most aspects of geology and geomechanics, much of these limitations surround the nature of geomaterials and their inherent variability. This heterogeneity occurs on almost all scales, from the grain size all the way through to the regional tectonic setting. Although some of this variability can be incorporated into the modelling techniques, much cannot, given the impracticality associated with considering every deviation from the idealized norm. This is particularly true in the case of the proposed
methodology, where data along the length of two multi-kilometer boreholes is considered. This can be most accurately described using the known/unknown descriptors. On one hand, there are the known-knowns, such as intact rock strength and the shape of borehole breakout. However there exists known-unknowns such as the scale effect on borehole wall strength and the relative influence of structure on the occurrence of not just breakout but also excavation scale breakout. Much of these known-unknowns are the focus of discussion in Section 8.3 on future work.

The potentially most detrimental set of conditions are from the unknown-unknowns. Much of these result from the ability of an investigation from the scale of a borehole to capture a relative sample size to fully describe the geological conditions. As the use of breakout geometry is the basis of the vast majority of this work, the presence and subsequent influence of sub-parallel discontinuities or elevated pore pressures is just an example of some things which can’t be quantified or considered. At best, such unknown-unknowns can only be deduced from erroneous results of breakout profiles. The following summary of limitations will focus on the known-unknowns, which can often be taken into consideration.

8.3 Future Work

There are conceivably multiple directions which can be pursued to further this work and bring the described methodologies to a level of confidence where it could ultimately be applied in practice. Given that the bulk of this work is at the stage of creating a process to evaluate borehole breakout, much of the future research should consider the validation of the proposed approaches. This can be done specifically as they apply to the Victoria Project itself, but also by branching out to different tectonic and geological settings, with inherently different mechanical behaviour in the borehole. The most apparent directions this research could take are:

- The evaluation of time dependency on borehole breakout, and the ideal time after borehole completion to undertake an ATV survey.
- Investigation of the influence of borehole water pressure on the progression of borehole breakout, as it applies to the frictional component of rock strength.
• A comparison of the results from the generalized modelling database with specific models run using the appropriate material properties for each rock type.

• The use of borehole breakout from other sites with existing stress state knowledge for methodology calibration.

• Review of a large volume of borehole breakout over an entire project to assess closeness of fit in the results from each borehole and predict spatial variability in stress in the lateral sense.

• Monitoring of shaft overbreak during development at the Victoria Project to validate model results of overbreak, as proposed in Chapter 7.

• Complete other situ stress measurement campaigns at the Victoria Project. This ideally would include further borehole breakout analysis on other boreholes or the use of alternative techniques such as overcoring or hydraulic-fracturing.

• Comprehensive sensitivity analysis of material assumptions as demonstrated in Chapter 7.

• Implementation of horizontal boreholes once development at The Victoria Project begins to verify results from the pilot holes.

8.4 Conclusions

During engineering design, the state of stress is one of the most influential parameters on the overall performance of an underground excavation. Whether this relates to mining infrastructure, nuclear waste storage facilities or deep alpine base tunnels, the role which stress plays cannot be overstated. Despite this, the input of stress is, in practice, often assumed from the regional tectonic setting or historic measurements. If the decision is made to complete some form of in situ testing, the cost associated with them often dictates a limited amount of tests, over a large area, which leads to uncertainty in the results.

Although the occurrence of borehole breakout has been recognized as an indicator of an over-stressed state since the 1970’s (Carr, 1974), relatively little work has been done to convey this into actual magnitudes of stress. This relates to many of the limitations discussed throughout this dissertation, relating to geological uncertainty, the scale effect of borehole strength and the subsequent numerical
methods to represent this behaviour. It is, however, these same limitations which drive much of the uncertainty associated with commonly used measurements such as overcoring and hydraulic fracturing.

The proposed methodology in this dissertation, presents a more continuous image of stress, which is more cost effective when the quantity of data is considered as compared to more conventional stress estimation techniques. The hopes of this approach are to not just complement the currently accepted methods but to also provide a substitute that can be more reliably done at earlier stages in a project’s life.

Potentially the most powerful result of this methodology is if borehole breakout is characterized at a high enough spatial frequency over a project. Given the continuity of stress estimate that is provided along the axis of the borehole, if multiple holes are assessed for breakout, this could develop a 3D model of stress. This is currently far from common practice in the mining industry, but would be an immense resource when creating the life-of-mine geomechanical model.

As more information is gained throughout a project’s development, this comprehensive stress model can be continuously updated to reflect in situ observations such as mine-induced seismicity or excavation damage mapping. This also applies to the prediction of overbreak by means of the modelling database. As this method relies on an accurate measurement of stress as an input, this further amplifies the importance of having confidence in the measuring technique and its associated result.

8.5 Contributions
The contributions to science and engineering that form the body of this thesis are highlighted through Chapters 3 to 7. All chapters work in one form or another, have been included in several conference proceedings and are being modified for the submission to scientific journals. These have been summarized below.
8.5.1 Fully Refereed Conference Papers and Presentations


8.5.2 Articles in Preparation


References


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ASTM (2013b) *Designation D4543-08: Standard Practices for Preparing Rock Core as Cylindrical Test Specimens and Verifying Conformance to Dimensional and Shape Tolerances*. West Conshohocken, USA.

ASTM (2013c) *Designation D7012-10: Standard Test Method for Compressive strength and elastic moduli of intact rock core specimens under varying states of stress and temperatures*. West Conshohocken, USA.


Itasca Consulting Group (2011) *FLAC version 7.0*.


Martin’s Thesis.


Figure A-1 - Comparison of fracture frequency per meter (FF/m) from both pilot holes with respect to core logging and acoustic televiewer survey.
## Appendix B

### Intact Lab Strength Testing Results

Table B-1 - Summary of Brazilian tensile strength testing completed on pilot on GT0020VCa.

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I- Intact failure  
PI- Partially intact failure  
S- Structural failure

### Table B-3 - Summary of various measurement types of Crack Initiation, as described in Chapter 3, Section 1.4.3.

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I- Intact failure  
PI- Partially intact failure  
S- Structural failure  
CI LS- Crack Initiation from Lateral Strain Reversal  
CI CVS_U- Crack Initiation from Crack Volumetric Strain Reversal  
CI CVS_max- Crack Initiation from Crack Volumetric Strain Maximum  
CI ITLS- Crack Initiation from Inverse Tangent Lateral Stiffness  
CI IPR- Crack Initiation from Instantaneous Poisson’s Ratio  
CI AE- Crack Initiation from Acoustic Emissions  
IC- Inconclusive
Table B-4 - Summary of various measurement types of Crack Damage Threshold, as described in Chapter 3, Section 1.4.3

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I- Intact failure  
PI- Partially intact failure  
S- Structural failure  
CD DAS- Crack Damage from Axial Strain Non-linearity  
CD RVS- Crack Damage from Volumetric Strain Reversa  
CD IYM- Crack Damage from Instantaneous Young’s Modulus  
CD AE- Crack Damage from Acoustic Emissions  
IC- Inconclusive
Appendix C

Model Validation

Elastic Solution (Analytical, RS2, FLAC3D)

Figure C-1 - Comparison of stress around an opening (x-axis), between the Kirsch solution, RS2 and FLAC3D. Results consider radial stress (\(\sigma_r\)) and tangential stress (\(\sigma_{\theta}\)).

Figure C-2 - Comparison of stress around an opening (y-axis), between the Kirsch solution, RS2 and FLAC3D. Results consider radial stress (\(\sigma_r\)) and tangential stress (\(\sigma_{\theta}\)).
Mohr-Coulomb (RS2, FLAC3D)

Figure C-3 - Comparison of stress around an opening (x-axis), between RS2 and FLAC3D using the Mohr-Coulomb constitutive model. Results consider radial stress (\(\sigma_r\)) and tangential stress (\(\sigma_{\theta}\)).

Figure C-4 - Comparison of stress around an opening (y-axis), between RS2 and FLAC3D using the Mohr-Coulomb constitutive model. Results consider radial stress (\(\sigma_r\)) and tangential stress (\(\sigma_{\theta}\)).
Figure C-5 - Comparison of stress around an opening, between the analytical plastic solution, RS2 and FLAC3D using the Hoek Brown constitutive model. Results consider radial stress ($\sigma_{r}$) and tangential stress ($\sigma_{\theta}$).
Appendix D

FLAC3D Breakout Database Code

;---------------------------------------------------------------------
;Master Breakout Database File
;---------------------------------------------------------------------

new

;-----Call Model Setup Files----------------
call mission_control.f3dat suppress ;Defines run type and file directory
call geometry.f3dat suppress ;Creates the model geometry and history locations
call material.f3dat suppress ;Assigns appropriate CWFS material properties
call ratiod.f3dat suppress ;Calls the files of in-plane stress and tangential stress ratios
call histtest.f3dat suppress ;Establishes history variables to be recorded
call Zonking.f3dat suppress ;Calls the file for modulus reduction on the borehole

;-----solver---------
call newloop.f3dat suppress ;Contains the fish function 'loopster' that goes through the
models at each stress state

def tp

    command
        @loopster
    endcommand

end

@tp

;---------------------------------------------------------------------
;Loop File for Database
;---------------------------------------------------------------------

def loopster

loop i (1,9) ;Loop the in-plane stress ratio from array "kratio"
    Kxy= parse(kratio(i),1)

    loop j (1,20) ;Loop the tangential stress from array "R-ratio"
        R= parse(R_ratio(j),1)
        stress_x= -1*(R*sig_ci)/(3*Kxy-1)
        stress_y=Kxy*stress_x
        stress_z= (stress_y+stress_x)/2

269
command
   ini sxx @stress_x
   ini syy @stress_y
   ini szz @stress_z

   step 500
   @_dozonk(7, 'excavate', 1); Solves the model using modulus reduction
endcommand

;-------Post Processing Data----------
name= 'Pilot Hole GT0020VC' + ' ' + 'Kxy=' + string(Kxy) + ' ' + 'R=' + string(R)
command
   save @name
   call histwrite.f3dat suppress ; Writes all the histories to .txt
   call datacrawler.f3dat suppress ; Writes the final model state to .txt
   call harry_plotster.f3dat suppress ; Creates figures of model results
endcommand

;-------Model Result Clearing----------
command
   model null range group Model
   ini stress 0,0,0,0,0,0
   ini vel 0,0,0
   ini disp 0,0,0
   ini state 0

   step 100
   @mat_prop ; Calls the material property file
endcommand
endloop
endloop
end