UNDERSTANDING AND PREDICTING EXCAVATION DAMAGE IN SEDIMENTARY ROCKS:
A CONTINUUM BASED APPROACH

By

Matthew Adrien Perras

A thesis submitted to the Department of Geological Sciences & Geological Engineering
in conformity with the requirements for the
Degree of Doctor of Philosophy

Queen’s University
Kingston, Ontario, Canada
(January, 2014)

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Abstract

The most widely accepted approach to long-term storage of nuclear waste is to design and construct a deep geological repository, where the geological environment acts as a natural barrier to radio nuclide migration. Sedimentary rocks, particularly argillaceous formations, are being investigated by many countries because of favorable isolating qualities (laterally continuous and low permeability) and the ability of self-sealing of fractures. Underground construction creates a damage zone around the excavation. The depth away from the excavation surface of the damage zone depends on the rock mass properties, the stress field, and the construction method. This research investigates the fracture development process in sedimentary rocks and evaluates continuum modelling methods to predict the damage zone dimensions.

At the laboratory scale, a complete classification system for samples of carbonate and siliciclastic rocks has been developed, with geotechnical considerations, which when applied narrows the variability of the mechanical properties. Using this system, crack initiation (CI) shows the most uniform range in each class, particularly for mud rocks. Tensile strength was found to be higher for the Brazilian method than Direct method of testing. Brazilian reduction to Direct values was found to be rock type dependent. Laboratory testing results are also influenced by the orientation of bedding.

Bedding and other structures were also found to influence the excavation behaviour as observed at the Niagara Tunnel Project in a mudstone and in excavations in the Quintner limestone of Switzerland. The conceptual stages of damage development and the potential fracture networks in sedimentary rocks are used to summarize the understanding of excavation damage developed in this thesis.

Using a continuum based modelling approach, a set of predictive damage depth curves were developed for the different excavation damage zones. This approach was found to be most sensitive to the tensile strength used as an input. Back analysis of the Niagara Tunnel Project and
forward prediction of the excavation damage around a shaft in the Queenston Formation are used to illustrate the importance of this research. The prediction methods were also applied to cut-off design analysis. This research has enhanced the understanding of excavation damage development in sedimentary rocks and provided a methodology to predict the dimensions of the excavation damage zones using a continuum based approach.
Co-Authorship

The thesis “Understanding and Predicting Excavation Damage in Sedimentary Rocks: A Continuum Based Approach” is the product of research conducted by the author, Matthew Adrien Perras. While scientific and editorial comments were made by Tom Lam, Helmut Wannenmacher, Ehsan Ghazvinian, Dr. Florian Amman, and Dr. Mark Diederichs, the written content is solely that of the author. Contributions from other colleagues in the form of conference papers which helped shape the content of this thesis, but do not directly contribute to the text are acknowledged in the contributions section of this thesis and the references section.
Acknowledgements

This research project has been made possible by the Nuclear Waste Management Organization (NWMO) of Canada, which sponsored this work financially through a Natural Sciences and Engineering Research Council of Canada Industrial Postgraduate Scholarship grant. The staff at NWMO have also provided support, feedback, and mentoring throughout this project and in particular I would like to thank Andrew Parameter, Monique Hobbs, Mark Jensen and Tom Lam for their efforts.

The continued use of the Niagara Tunnel Project (NTP) data set and for access to core samples from the Niagara Region has been made possible by Ontario Power Generation (OPG) and Hatch Energy. A special thank you is due to Rick Everdell (OPG), Warren Hoyle (Hatch), Ralph Serluca (Hatch), Stephen Rigbey (formerly Hatch, now BC Hydro) and David Besaw (Hatch) for their assistance with respect to aspects of this research related to the NTP and the Niagara Region. In particular Rick Everdell and David Besaw have assisted greatly with the publications associate with the NTP.

Laboratory testing has been conducted at Queen’s University in The Robert M. Buchan Department of Mining, the Royal Military College (RMC) of Canada, and at CANMET Mining and Mineral Sciences Laboratories a division of Natural Resources Canada. A particular thank you is due to the staff at RMC for all their assistance with setting up the testing equipment, the valuable discussions on instrumentation and help during testing. As well the staff at CANMET have been invaluable at helping me understand the care and quality needed to get great testing results. A special thank you is due to Denis Labrie (CANMET) for supporting this research and encouraging me along the way. Also a special thank you is due to Jeff Oke (Queen’s / RMC) and Ehsan Ghazvinian (Queen’s) for their assistance with the laboratory testing and numerical modelling and the countless discussions on these topics. In fact there are many Queen’s Geomechanics Group members who should be listed, both past and present, who have been there
over the years to listen, give advice, lend a helping hand, and generally support and encourage me during this time of my life. These include; Drs. Kathy Kalenchuk, Nicholas Vlachopoulos, Matt Lato, and Connor Langford, as well as Anna Crockford, Dani Delaloye, Steve Gaines, Colin Hume, Dave Gauthier, Cortney Palleske, Felipe Duran, Gabe Walton, Michelle van der Pouw Kraan, Jenn Day, Kiarash Farahmand, Chrysothemis Paraskevopoulou, and Cara Kennedy.

Other colleagues have also been instrumental in helping with this research. In particular, Dr. Florian Amann and Helmut Wannenmacher assisted with access to sites and data for this research, reviewed this work, encouraged me along the way, and became great friends in the process.

My supervisor, Dr. Mark Diederichs, began encouraging me towards research in my Undergraduate degree (even before I realized it). He helped me get an excellent summer job with Suncor and again my first full time job with Hatch. When I graduated with my BScEng he told me to keep my ears open for opportunities to work on the Niagara Tunnel Project, which I ended up working on for two years. My time working at the tunnel inspired me to pursue an understanding of how the damage process developed in the Queenston Formation and this led to completing my Masters and now my PhD. Mark has provided invaluable guidance, support and motivation. He has shared with me a wealth of knowledge and experience and provided many opportunities, which would have otherwise not been possible. Mark, thank you for all the support, memories, and friendship.

My family has also encouraged me from day one, being supportive of whatever my pursuits maybe. My mother and father, Sharon and Andre Perras, my sister and brother-in-law, Jenny and Mike and my niece Mackenzie, thank you for listening to me and encouraging me along the way.

To wife Andrea, words fall short of the thanks that I would try to express to you. Simply, thank you for being who you are and being there for me. You certainly do deserve your PhT.
Statement of Originality

(Required only for Division IV Ph.D.)

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

(Matthew Adrien Perras)

(December, 2013)
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<tbody>
<tr>
<td>A</td>
<td>Cross sectional area of sample</td>
</tr>
<tr>
<td>a</td>
<td>Excavation radius</td>
</tr>
<tr>
<td>$a$</td>
<td>Hoek-Brown material constant</td>
</tr>
<tr>
<td>BTS</td>
<td>Brazilian Tensile Strength</td>
</tr>
<tr>
<td>CC</td>
<td>Crack Closure</td>
</tr>
<tr>
<td>CD</td>
<td>Crack Propagation</td>
</tr>
<tr>
<td>CI</td>
<td>Crack Initiation</td>
</tr>
<tr>
<td>COV</td>
<td>Coefficient of Variation</td>
</tr>
<tr>
<td>D</td>
<td>Diameter</td>
</tr>
<tr>
<td>$D_e^2$</td>
<td>Equivalent core diameter</td>
</tr>
<tr>
<td>DTS</td>
<td>Direct Tensile Strength</td>
</tr>
<tr>
<td>$E_{\text{beam}}$</td>
<td>Rock beam modulus</td>
</tr>
<tr>
<td>$E_i$</td>
<td>Young’s Modulus</td>
</tr>
<tr>
<td>$E_{\text{rm}}$</td>
<td>Rock mass modulus</td>
</tr>
<tr>
<td>$F_a$</td>
<td>Applied force</td>
</tr>
<tr>
<td>$f_{\text{BTS}}$</td>
<td>Reduced Brazilian Tensile Strength</td>
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<tr>
<td>$I_{s50}$</td>
<td>Point Load Index</td>
</tr>
<tr>
<td>$K_f$</td>
<td>Stress concentration factor</td>
</tr>
<tr>
<td>$K_{\text{Hh}}$</td>
<td>Maximum Horizontal to Minimum horizontal Stress Ratio</td>
</tr>
<tr>
<td>$K_{\text{hv}}$</td>
<td>Minimum horizontal to Vertical Stress Ratio</td>
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<td>$K_n$</td>
<td>Normal stiffness of a joint element</td>
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<td>$K_o$</td>
<td>Vertical to Maximum Horizontal Stress Ratio</td>
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<td>$K_s$</td>
<td>Shear stiffness of a joint element</td>
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<td>L</td>
<td>Sample length</td>
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<td>$m_{\text{b}}$</td>
<td>rock mass Hoek-Brown material constant</td>
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<td>$m_d$</td>
<td>Dilation parameter</td>
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<td>intact Hoek-Brown constant</td>
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<td>MR</td>
<td>Modulus Ratio</td>
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<td>Modified Stability Number</td>
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<td>$P$</td>
<td>Failure load for point load index testing</td>
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<td>Water pressure</td>
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<tr>
<td>r</td>
<td>Radius of damage</td>
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<tr>
<td>Symbol</td>
<td>Description</td>
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<tr>
<td>R</td>
<td>Sample radius</td>
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<tr>
<td>$R^2$</td>
<td>Coefficient of determination</td>
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<td>$s$</td>
<td>Hoek-Brown material constant</td>
</tr>
<tr>
<td>SL</td>
<td>Tunnel Spring Line - located at mind height on the side wall</td>
</tr>
<tr>
<td>St. Dev.</td>
<td>Standard Deviation</td>
</tr>
<tr>
<td>t</td>
<td>Sample thickness</td>
</tr>
<tr>
<td>T</td>
<td>Tensile Strength</td>
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<tr>
<td>UCS</td>
<td>Unconfined Compressive Strength</td>
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<td>$V_p$</td>
<td>P-wave velocity</td>
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<tr>
<td>$V_s$</td>
<td>S-wave velocity</td>
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<tr>
<td>wt%</td>
<td>Weight percentage</td>
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<tr>
<td>$\alpha$</td>
<td>Angular width of loading</td>
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<td>$\varepsilon_a$</td>
<td>Axial strain</td>
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<tr>
<td>$\varepsilon_l$</td>
<td>Lateral strain</td>
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<td>$\varepsilon_{vol}$</td>
<td>Volumetric strain</td>
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<td>$\nu$</td>
<td>Poisson’s Ratio</td>
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<td>Maximum principal stress</td>
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<td>Maximum tangential stress at an excavation boundary</td>
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<td>$\Phi$</td>
<td>Friction angle</td>
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<tr>
<td>$\Psi$</td>
<td>Dilation angle</td>
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<td>3PB</td>
<td>Three Point Bending test</td>
</tr>
<tr>
<td>4PB</td>
<td>Four Point Bending test</td>
</tr>
<tr>
<td>AE</td>
<td>Acoustic Emission</td>
</tr>
<tr>
<td>AECL</td>
<td>Atomic Energy of Canada</td>
</tr>
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<td>DGR</td>
<td>Deep Geological Repository</td>
</tr>
<tr>
<td>DISL</td>
<td>Damage Initiation and Spalling Limit</td>
</tr>
<tr>
<td>EDZi</td>
<td>Inner Excavation Damage Zone</td>
</tr>
<tr>
<td>EDZo</td>
<td>Outer Excavation Damage Zone</td>
</tr>
<tr>
<td>EDZs</td>
<td>Excavation Damage Zones, including EDZi, EDZo, and HDZ</td>
</tr>
<tr>
<td>EIZ</td>
<td>Excavation Influenced Zone</td>
</tr>
<tr>
<td>GSI</td>
<td>Geological Strength Index</td>
</tr>
<tr>
<td>HDZ</td>
<td>Highly Damaged Zone</td>
</tr>
<tr>
<td>ITLS</td>
<td>Inverse Tangent Lateral Stiffness</td>
</tr>
<tr>
<td>L&amp;ILW</td>
<td>Low and Intermediate Level Waste</td>
</tr>
<tr>
<td>LAM</td>
<td>Laminated Anisotropy Method</td>
</tr>
<tr>
<td>NTP</td>
<td>Niagara Tunnel Project</td>
</tr>
<tr>
<td>OPG</td>
<td>Ontario Power Generation</td>
</tr>
<tr>
<td>RMR</td>
<td>Rock Mass Rating</td>
</tr>
<tr>
<td>SAB GS</td>
<td>Sir Adam Beck Generating Station</td>
</tr>
<tr>
<td>TBM</td>
<td>Tunnel Boring Machine</td>
</tr>
<tr>
<td>UBJT</td>
<td>Ubiquitous Joint Double Yield Model</td>
</tr>
<tr>
<td>URL</td>
<td>Underground Research Laboratory</td>
</tr>
<tr>
<td>WIPP</td>
<td>Waste Isolation Pilot Plant</td>
</tr>
</tbody>
</table>
Chapter 1: Introduction

1.1 Purpose of this Research
In Canada, nuclear energy research began in the 1940s and since that time a total of 2.4 million cubic meters of radioactive waste has been generated, including high-, intermediate-, and low-level waste (LLRWMO 2012). Currently this inventory is stored at surface. The generally accepted long-term storage solution is in a deep underground repository, which utilizes the natural geology as a barrier to isolate the repository from the near surface hydrogeological environment and biosphere. The successful deep underground repository or more commonly deep geological repository (DGR) must isolate the nuclear waste for upwards of one million years due to the long-lived contaminants. The concept is illustrated in Figure 1.1 with vertical emplacement boreholes for crystalline rocks and horizontal tunnels for sedimentary rocks at a horizon typically in the order of 500m below the ground surface.

The majority of the nuclear waste in Canada is located in Southern Ontario, which is underlain by sedimentary rocks, as shown in Figure 1.2. The sedimentary sequence is comprised of carbonates, shales, evaporates, and sandstones. The shales are an excellent isolating layer between the basin brines at depth and the near surface aquifers (Mazurek 2004). The sedimentary sequence in Ontario has the added advantage of a strong, low permeability, argillaceous carbonate sequence below the shale sequence (Figure 1.2). The isolating shale is an important element of the DGR concept as it keeps ground water velocities and flux to a minimum (Russell and Gale 1982). The stability of repository excavations, particularly around access shafts passing through the isolating shale, are therefore a key consideration in the design process.
Figure 1.1: Conceptual deep geological repository arrangement for crystalline (vertical placement of waste container) and sedimentary (horizontal placement) rocks (modified from Russell 2011).
Figure 1.2: Geological cross section across the Algonquin Arch (see inset) showing generalized geology of both the Michigan and Appalachian Sedimentary Basins. Cross section has extreme vertical exaggeration (modified from Armstrong and Carter 2006).
The construction of underground excavations can cause damage to the rock mass to varying degrees depending on the excavation method employed, structure of the rock mass, and changes in the stress around the excavation, for example. The density of fracture damage around the excavation will depend on the in-situ conditions, although it will generally result in zones with decreasing fracture density farther away from the excavation boundary, as illustrated in Figure 1.3. The excavation damage zones (EDZs) will change the permeability of the rock mass locally creating a flow pathway, which potentially will be continuous along the axis of the excavation. To mitigate the flow along the EDZs a slot can be cut into the rock mass (perpendicular to the excavation boundary) and filled with a low permeability material, such as bentonite. This slot is called a cut-off and is illustrated in Figure 1.3.

The purpose of this study is to understand the development of EDZs in sedimentary rocks, particularly mud rocks and carbonates, and apply this understanding in numerical models to predict the dimensions of the EDZs. There are four main goals of this study, as follows:

- to examine and classify mechanics and characteristics of sedimentary rocks at the laboratory and excavation scale, with a particular focus on excavation damage
- to investigate critical mechanical properties of sedimentary rocks,
- to understand the relationships between continuum model outputs and excavation damage zone characteristics for prediction of the damage dimensions, and
- to determine and describe the sensitivity of prediction, verification and design models to material properties, excavation geometry, and boundary conditions.

This research will build on the experience gained from studying excavation damage in granitic and gneissic rock masses, such as the pioneering work at Canada’s underground research laboratory (URL) in Pinawa, Manitoba or Sweden’s Apso URL. The concepts and characteristics of excavation damage development in granitic and gneissic rock masses will briefly be reviewed in the following sections by way of an introduction.
Figure 1.3: Schematic illustration of (top) excavation damage around and along a circular tunnel and (bottom) relationship between distance from excavation surface and permeability (modified from ANDRA 2005).
1.2 Underground Research Laboratories
The term Excavation Damage Zone (EDZ) was used and first studied in detail for nuclear waste storage at the Stripa mine site, in Sweden (Carlsson 1986). Initially research on damage around underground excavations focused on brittle hard rocks, such as crystalline rock masses (Carlsson 1986), with some countries also investigating old salt mines in the 1960s (US Department of Energy 2000). Many countries are now studying the EDZ in sedimentary rocks, particularly argillaceous rocks, as potential hosts for the long-term management of nuclear waste. A list of current and past URLs, as well as active investigation sites is shown in Table 1.1. Based on the current research and investigation sites 46% are in granite, 17% are in salt, 21% argillaceous rocks, and 13% are in other rocks or soils.

The purpose of such URLs is to characterize the rock mass, to develop and test technology associated with underground disposal and to demonstrate the feasibility of underground storage in the rock mass in question or similar rock masses. These areas of study support three main elements that need to be defined and developed for an underground waste repository to be developed, including:

- Facility sitting and disposal system design,
- Underlying scientific and engineering support, and
- Evaluation of safety.

Research on the development of the EDZ from crystalline rock masses is a useful starting point for understanding the development in sedimentary rocks. Early studies at Canada’s URL, near Pinawa Manitoba, and at Sweden’s Äspö URL document the brittle spalling and fracture network development in crystalline rock masses. Building on this research, as well as observations and numerical analyses conducted as part of this research, a comprehensive understanding of EDZ in sedimentary rocks will be presented.
Table 1.1: The prominent past, active, temporary or proposed underground research laboratories (URL) or deep geological repository investigation sites (DGR) for the storage of nuclear waste (modified from IAEA 2001).

<table>
<thead>
<tr>
<th>Country</th>
<th>Facility Name</th>
<th>Location</th>
<th>Geology</th>
<th>Depth (m)</th>
<th>Period</th>
</tr>
</thead>
<tbody>
<tr>
<td>Belgium</td>
<td>HADES URF</td>
<td>Mol</td>
<td>Soft clay</td>
<td>223</td>
<td>Since 1980</td>
</tr>
<tr>
<td>Canada</td>
<td>OPG DGR</td>
<td>Kincardine</td>
<td>Argillaceous limestone</td>
<td>680</td>
<td>Investigations</td>
</tr>
<tr>
<td>Canada</td>
<td>AECL</td>
<td>Pinawa, Manitoba</td>
<td>Granite</td>
<td>420</td>
<td>1984 -</td>
</tr>
<tr>
<td>Czech Republic</td>
<td>Shaft 16</td>
<td>Pribram</td>
<td>Granite</td>
<td></td>
<td>Late 1990’s</td>
</tr>
<tr>
<td>Finland</td>
<td>Onkalo tunnel</td>
<td>Olkiluoto</td>
<td>Granite</td>
<td>400</td>
<td>Since 1993</td>
</tr>
<tr>
<td>France</td>
<td>Fanay</td>
<td>Fanay Augeres</td>
<td>Granite</td>
<td>1980 – 1990</td>
<td></td>
</tr>
<tr>
<td>France</td>
<td>Amelie</td>
<td>Amelie</td>
<td>Salt</td>
<td>1986 – 1994</td>
<td></td>
</tr>
<tr>
<td>France</td>
<td>Tournemire tunnel</td>
<td>Tournemire</td>
<td>Shale</td>
<td>250</td>
<td>Since 1990</td>
</tr>
<tr>
<td>France</td>
<td>Meuse/Haute Marne</td>
<td>Bure</td>
<td>Shale</td>
<td>~500</td>
<td>Since 2000</td>
</tr>
<tr>
<td>Germany</td>
<td>Schacht Asse II</td>
<td>Asse</td>
<td>Salt</td>
<td>750</td>
<td>1977 – 1995</td>
</tr>
<tr>
<td>Germany</td>
<td>Gorleben</td>
<td>Gorleben</td>
<td>Salt</td>
<td>1997</td>
<td>Since 1997</td>
</tr>
<tr>
<td>Germany</td>
<td>ERAM</td>
<td>Morsleben</td>
<td>Salt</td>
<td>630</td>
<td>Since 1981</td>
</tr>
<tr>
<td>Germany</td>
<td>Schacht Konrad</td>
<td>Konrad</td>
<td>Shale</td>
<td>800</td>
<td>Since 1980</td>
</tr>
<tr>
<td>Hungary</td>
<td>Pecs</td>
<td>Pecs</td>
<td>Shale</td>
<td>1995-1999</td>
<td></td>
</tr>
<tr>
<td>Hungary</td>
<td>Bataapati</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Japan</td>
<td>Tono mine</td>
<td>Tono</td>
<td>Sandstone</td>
<td>1986</td>
<td></td>
</tr>
<tr>
<td>Japan</td>
<td>Honorobe</td>
<td>Honorobe</td>
<td>Sedimentary Development</td>
<td></td>
<td></td>
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<tr>
<td>Korea</td>
<td>Gyeongju</td>
<td></td>
<td></td>
<td>80</td>
<td></td>
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<tr>
<td>Sweden</td>
<td>SFR</td>
<td>Forsmark</td>
<td>Granite</td>
<td>50</td>
<td>Temporary</td>
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<td>Sweden</td>
<td>Forsmark</td>
<td>Forsmark</td>
<td>Granite</td>
<td>450</td>
<td>Proposed</td>
</tr>
<tr>
<td>Sweden</td>
<td>Åspö</td>
<td></td>
<td>Granite</td>
<td>450</td>
<td>Since 1990</td>
</tr>
<tr>
<td>Switzerland</td>
<td>GTS</td>
<td>Grimsel</td>
<td>Granite</td>
<td>400</td>
<td>Since 1983</td>
</tr>
<tr>
<td>Switzerland</td>
<td>Mont Terri tunnel</td>
<td>Mont Terri</td>
<td>Shale</td>
<td>400</td>
<td>Since 1995</td>
</tr>
<tr>
<td>UK</td>
<td>RCF</td>
<td>Sellafield</td>
<td>Tuff</td>
<td>- 1997</td>
<td></td>
</tr>
<tr>
<td>USA</td>
<td>WIPP</td>
<td>New Mexico</td>
<td>Salt</td>
<td>655</td>
<td>Since 1982</td>
</tr>
<tr>
<td>USA</td>
<td>ESF Project</td>
<td>Yucca Mt.</td>
<td>Tuff</td>
<td>200-300</td>
<td>1993 – 2012</td>
</tr>
</tbody>
</table>
1.3 Pioneering EDZ Concepts from Granitic and Gneissic Rocks

Some of the most notable studies in granitic rock masses have been carried out at Canada’s underground research laboratory operated by Atomic Energy of Canada Ltd. located in southeastern Manitoba and the Äspö hard rock laboratory (HRL) in Sweden. From these studies in granitic rocks an understanding of brittle rock mass behaviour, its fracture characteristics, and how to simulate the behaviour numerically was understood more clearly (Martin 1997, Read 2004, Martino and Chandler 2004, Diederichs 2007), among many other findings.

Figure 1.4: Canada’s URL near Pinawa, Manitoba, Canada, showing generalized geology relative to the different levels of the URL (modified from Chandler 2003 and Read 2004).
Excavation for Canada’s URL began in 1984, with a shaft excavated into the batholith of Lac du Bonnet granite (Read 2004). Some of the notable studies regarding brittle behaviour and excavation induced damage from Canada’s URL have been summarized by Martino and Chandler (2004) and Read (2004). The excavation response can be generally separated into two broad regions; the moderately to highly fractured zones above the 300 level (see Figure 1.4) and the sparsely fractured zone starting around the 300 level. At the 300 level the stress magnitudes are significantly higher than those above (Read 2004), which results in significant excavation induced damage.
Figure 1.6: Schematic representation of the progressive failure process resulting in a notch geometry commonly associated with brittle spalling behaviour (modified from Read 2004, after Martin 1993).
Monitoring and characterization of the shaft excavation in the upper fractured granite at the URL indicated that subvertical jointing dominated the mechanical response of the rock mass (Read 2004). In the sparsely fractured granite, stress induced fractures dominated. They typically formed a V-shaped notch when the axis of the tunnel was oriented perpendicular to the maximum stress orientation (Martin 1997). Observations show that the v-shaped notch was continuous along the tunnel length (Figure 1.5a), although in places the shape changed with variations in geology (Martino and Chandler 2004). The notch formed in the massive granite with little evidence of being aided by pre-existing structure (Figure 1.5b). The notch developed as a series of thin, notch surface parallel fractures (Figure 1.5c) as a result of the stress path around the excavation. Read (2004) stated that the notch formation was more evident in the granite than in the granodiorite lithology.

The notch formation process has been described by Martin (1993) as a progressive failure process, the stages of which are shown in Figure 1.6. First, crack initiates in the area of maximum tangential stress around the excavation boundary. Secondly, the cracks coalesce into a small process zone, which is commonly an area of intense crushing of the rock mass. The process zone was reported to range between 50-100 mm by Martin (1993). Thirdly, fractures begin to propagate away from the process zone following the stress contours around the developing notch. This results in thin slabs of rock, typically only as thick as the grain size (Martin 1997), which progressively get thicker as the notch deepens. As the notch develops, the stress contours also change causing the fracture orientation to change and remain parallel to the notch surface. The length of the excavation surface over which the maximum tangential stress acts is longer prior to notch development and as the notch forms this length decreases until the notch stabilizes due to the increased confining stress at the notch tip.
Figure 1.7: Ovaloid excavation geometry with the long axis a) horizontal and b) inclined to reduce the maximum tangential stress at the excavation boundary (modified from Read 2004).
At this point the maximum stress is focused at the notch tip and the stress acting on the sides of the notch are reduced. The notch eventually reaches a point of equilibrium (Stage 4 in Figure 1.6). According to Read (2004) the notch stability was extremely sensitive to minor changes in stress, humidity and temperature at the URL.

The development of the notch at the URL could be minimized or eliminated by changing the shape or orientation of the excavation with respect to the stress field. The shape alone (Figure 1.7a) was insufficient to minimize damage. It was found that an ovaloid shape, long axis parallel to the maximum stress (Figure 1.7b) resulted in the best stress distribution (Read 2004).

Excavation induced damage inside the rock mass around the excavations at Canada’s URL were investigated by a number of methods, as reported by Martino and Chandler (2004). The fracture pattern within the spall notch reflects the stress flow path, as previously mentioned. Detailed visual studies of the fracture pattern outside of the notch, from Canada’s URL, have not been studied or published to the author’s knowledge. However; velocity and acoustic emission (AE) measurements suggest that the fractures should generally be aligned parallel to the excavation surface (Meglis et al. 2005). An exception to the above finding was detected in the sidewalls where radially aligned (normal to the tunnel wall) fractures where detected as low amplitude AE events, but without visible damage (Meglis et al. 2005). Presumably these are isolated micro cracks or movement on existing flaws in the rock mass that have not coalesced to permit a portion of the rock mass to fall away from the excavation surface.

Extensive damage characterization work has been conducted in Sweden and the observations related to the fracture network are briefly examined for comparison to those in sedimentary rocks, which will be discussed later. Jonsson et al. (2009) presented the main projects relating to the evolution of the damage around the excavations at the Åspö URL.
Figure 1.8: Fractures generated in a drill and blast tunnel examined from cut-away slots at the Äspö URL, with half barrel rounds indicated where visible (combined from Olsson et al. 2004 and Emsley et al. 1997).
A blasting experiment concluded that normal blasting could induce up to a 1.7 m thick EDZ and that cautious/controlled blasting could reduce the thickness to 1 m (maximum reported) in the granitic rocks (Jonsson et al. 2009). Further reduction of the zone thickness could be achieved by utilizing secondary blasting (slashing).

The ZEDEX project also investigated the evolution of the EDZs and the EIZ, which is referred to by Jonsson et al. (2009) as the disturbance zone. He compared drill and blast methods with tunnel boring machine excavation methods and found that the EDZ in the drill and blast tunnel was larger. The average EDZ in the drill and blast was 0.3 m deep and for the TBM excavation is was found to be 0.03 m (Emsley et al. 1997). The EDZ was determined using a variety of seismic methods to determine the wave velocity close to the tunnel wall (Jonsson et al. 2009). Another significant finding by Jonsson et al. (2009) was that there was no difference in the EIZ between the two excavation methods based on acoustic emission activity.

The EDZ fracture pattern for the drill and blast excavation was extensively investigated by cutting slabs of rock from the wall and floor, as shown in Figure 1.8. Dye was injected into the saw cuts around the slabs so it would seep into the cracks that were connected to an inner damage zone, which helped to identify the crack locations (Jonsson et al. 2009). The fracture patterns associated with the blasting can be seen to emanate from the half barrel rounds indicated in Figure 1.8. They either deviate parallel to the excavation wall (Figure 1.8, left side top and bottom) or propagate toward existing fractures (Figure 1.8, right, bottom). The longer induced fractures tend to align with the excavation wall and the shorted induced fractures are angled slightly upwards. Close to the excavation surface there is evidence of an interconnected network of fractures. The interconnectedness quickly dissipates into the excavation. It should be noted that the stress levels at the Äspö URL are low enough that a stress induced notch does not develop.
Studies at the TBM excavation were unable to reveal visible (macro-scale) fractures. The drill and blast network can be considered a loosely connected network of macro-scale fractures. It was reported that this network is also not well connected in the direction of the excavation axis (longitudinally).

This past experience shows that in granitic rocks the fracture network ranges from:

- closely spaced fractures in the shape of a V-notch with longitudinal connectivity,
- to loosely connected (laterally and longitudinally) macro-scale fractures,
- to minimal macro-scale fractures,

which depends on the stress and strength at the site of investigation.

1.3.1 Observations from Existing URLs

As part of this research project a number of active URL’s were visited to understand the current state of research and observed firsthand the excavation behaviour in both granitic and sedimentary rocks. The URL’s which were visited are shown in Figure 1.9 and also included is the low and intermediate level radioactive operational waste storage facility, SRF, in Forsmark, Sweden.

The SRF was commissioned in 1988 to store low and intermediate level radioactive operational waste, which includes protective clothing, cleaning supplies (mop heads, rags, etc.) and used parts from the power plants which have not been in direct contact with the fuel. The SRF was constructed to hold a capacity of 63 000 m³ and in early 2014 the Swedish Nuclear Fuel and Waste Management Company (SKB) expects to submit an application for a license to extend the SRF facility (SKB 2014). The existing facility is 60 m below the sea floor and the proposed extension is planned to be 120 m below the sea floor, both in granites and granodiorites. This is an operating facility and not used for research experiments.
Both the Äspö and Grimsel URL’s were visited as part of this research project. The Äspö URL is a purpose built research facility, meaning that it was entirely constructed for research activities. The facility is approximately 460 m deep and located in granitic and dioritic rocks (Martin et al. 2001). Experiments related to underground storage of nuclear waste have been ongoing since construction of the facility began in 1990. The Grimsel URL, in Switzerland, is also located in granitic rocks. In contrast with the Äspö facility, Grimsel was constructed off an existing tunnel system for hydropower generation in 1983 (Grimsel 2014). The facility is around 450 m below the ground surface. Äspö and Grimsel are among the most active URL’s in granitic rocks.

In sedimentary rocks, the Mont Terri and Meuse/Haute-Marne URL’s were visited. The Mont Terri facility is located in the Canton of Jura, Switzerland. It is an expansion from a security tunnel for the A16 motorway tunnel. Research niches were excavated starting in 1996 and further expansion of the facility took place in 1998, 2003, 2004, and 2008 (Mont Terri 2014). The facility is located in the Opalinus clayshale at a depth of approximately 300 m. The Meuse/Haute-Marne facility was purpose built for research activities near the town of Joinville, France. Construction of the URL began in 1999 and the main investigation horizon is at 490 m below the ground surface in the Callovo-Oxfordian unit, an argillite, approximately 200 m thick.

The difference in excavation behaviour of the granitic rock masses and the sedimentary rock masses is shown in Figure 1.9. The fracture patterns in the granitic rock masses follow the excavation boundaries, even for example at Grimsel (Figure 1.9, number 3) around the corner at an intersection of two tunnels. In the sedimentary examples there is evidence of the influence of the bedding plane orientation on the damage pattern around the excavation. At Mont Terri the
inclined bedding shows signs of shearing and at Meuse/Haute-Marne the horizontal bedding creates a flat top of the originally circular excavation (Figure 1.9, numbers 4 and 5, respectively).

Figure 1.9: A storage facility for Low and Intermediate Level Waste at Forsmark, Sweden (1 courtesy of skb.se), and various observations from Underground Research Laboratories in Sweden (2), Switzerland (3,4) and France (5). The observations are discussed in the text.
The common anisotropy of sedimentary rocks influences the excavation behaviour and the challenges of underground excavation in sedimentary rocks are different than in granitic rocks.

1.4 Underground Excavation Challenges in Sedimentary Rocks

Similar to granitic rocks, excavation damage in sedimentary rocks is stress and strength dependent. There are added challenges to excavating in sedimentary rocks which are less likely in granitic rocks and some of these challenges are rock type dependent.

For instance salt deposits are highly soluble when exposed to fresh water and therefore careful shaft access seal design must be considered to be a key component of the repository to prevent fresh water access during construction, operation and closure. Fresh water inflow can also cause swelling of some clay or shale deposits. In some cases a thin layer of shotcrete can minimize the swelling process by exerting a small amount of confinement and creating a barrier.

Clay deposits, such as the Boom clay in Belgium (see Table 1.1) present constructability challenges which require extensive support to advance the excavation. Large damage zone radii are expected in the Boom clay from Belgium, however self-sealing is expected (Mertens et al. 2004). Self-sealing is the ability of the rock mass to seal open fractures and reduce the permeability to near undamaged levels. This differs from self-healing, which is defined as sealing with loss of memory of the pre-healing state (Tsang and Bernier 2005). Therefore a healed fracture will not be a preferred site for new fracturing just because of its history. A sealed fracture is not necessarily healed, however, a healed fracture is also sealed (Bock et al. 2010).

The installation of steel support elements will generate gas pressures as the steel deteriorates over time. These gas pressures may be able to expand existing fracture networks or
previously sealed networks and increase the permeability in the damage zone. The more support required, the more gas generation will occur.

Sedimentary rocks are also typically anisotropic in terms of strength and stiffness. Work by the author (2009) demonstrated that when the plane of anisotropy parallels the axis of the excavation and the principal stress orientation, large over break can occur depending on the stiffness. In this case, changing the excavation orientation relative to the stress field can improve the excavation response, similar to that of the granitic excavations discussed previously.

Slate, clay, shale and mudstones are typically weaker rock masses than granitic rock masses. Limestones fall in between mudstones and granites in terms of strength and the anisotropic strength and stiffness is less pronounced than in shales (Ghazvinian et al. 2013). In Canada, the Cobourg argillaceous limestone is the proposed repository horizon for Low and Intermediate level nuclear waste. The argillaceous interbeds of the Cobourg can provide some measure of self-sealing capacity to the rock mass, although this will depend on the lateral extent of the interbeds. Self-sealing is a physio-chemical process by which fractures, natural or induced, become less conductive with time and no longer form a preferential flow pathway for radionuclides (Bock et al 2010). This process differs from self-healing, which implies that the deformation and strength properties of the healed material are now identical to the undisturbed material, which may not be the case for all self-sealed fractures (Bock et al 2010). Conceptually the argillaceous nature can also help to minimize the extent of the excavation damage by absorbing the fracture propagation energy. However there is little previous excavation experience in similar argillaceous limestones at comparable depths to the proposed repository horizon.
1.5 Character of EDZs

The definitions of the different zones around an underground excavation, where the properties of the rock mass have been altered from the original state, have received much attention, particularly for underground storage of nuclear waste. Lanyon (2011) reviews early sources for definitions of the different zones around underground excavations.

The terminology for the different damage zones used in this thesis are modified from those of Lanyon (2011) to reflect both the impact of the damage intensity on flow properties as well as to indicate the more useful numerical indicators for prediction of the different zones induced by the excavation process, as in Figure 1.10. The revised nomenclature is as follows:

- The Excavation Influence Zone (EIZ) - formerly the Excavation disturbed Zone (EdZ)
- The Excavation Damaged Zone (EDZ) - includes only distributed damage
- Highly Damaged Zone (HDZ) – formerly the Excavation Fracture Zone

The EDZ and HDZ are referred to as the Excavation Damage Zones (EDZs) and refer to the damage induced by the excavation process, including stress induced fractures.

Figure 1.10: Damage zones for massive and jointed ground around a circular tunnel.
The EIZ is a zone of stress change and/or elastic strain influence. The material in question behaves in an elastic manner and no plastic yielding is induced. In more porous rocks the stress/strain influence can result in changes in pore geometry and scale and therefore result in possible hydromechanical and geochemical modifications. Therefore the EIZ permeabilities will be only affected in high porosity materials (change in open pore geometry). Large stress changes may alter grain boundaries in low porosity rocks or effective joint apertures in rock masses but the influence on permeability should be minimal.

The outer limit of the EIZ can be defined in numerical models as the point where the stress state returns to near pre-excavation levels and orientation. This is typically in the tens of meters at the size of typical nuclear waste repository galleries. The outer boundary typically is of little or no concern for engineering design considerations of a nuclear waste repository.

The EDZ is a zone of physical inelastic damage with hydro-mechanical modifications inducing changes in flow and transport properties. Rock mass permeabilities can increase by a factor of 10 times the background value near the outer boundary to as much as 1000 times the background value at the inner excavation boundary (ANDRA 2005). Bastiaens et al. (2007) stated that an increase in the hydraulic conductivity by less than a factor of ten is not significant and this measure has been used as the dividing line between the EIZ and EDZ. The damage in the EDZ is in the form of grain scale fractures, minor interface dilation, and discontinuous internal damage. This damage is typically non-connected in low porosity rocks, at the boundary with the EIZ, and the nature of the damage is related to the internal structure of the intact rock. At the boundary with the HDZ, the micro-scale damage has become more interconnected. As a result of this research, the EDZ can be divided into two gradational zones, the EDZi (inner EDZ) and the
EDZo (outer EDZ). The boundary between these two zones can be defined as an engineering target for porosity/permeability/diffusivity increase.

The influence on permeability and diffusivity can be expressed as a relative change from the undamaged values of the rock mass. The permeability increase, for example can be related to an increase in volumetric strain (expansion).

The EDZ is influenced primarily by induced stresses and excavation geometry and is unlikely to be directly modified by changes to the support system or artificial remediation, although restriction of the HDZ and confinement imposed on the excavation surface will serve to reduce the extent of the EDZ.

In this work it has been shown that the damage in the EDZ is easily identified in modelling by yield indicators (stresses exceeding damage criterion) accompanied by limited plastic strain and/or limited internal stress reduction. The boundary with the EIZ is normally sharp. The limits of the EIZ in numerical models will be sensitive to the selection of the yield function under confinement (elevated mean stress). For the same unconfined compressive stress (controlling the onset of damage at the excavation boundary) a linear or non-linear relationship between yield and confining stress will result in different EDZ extents. The use of several criteria is advised to determine a likely range for EDZ extent.

As stated previously, the EDZ is a zone which is stress and strain induced and the extent of the EDZ will not be significantly influenced by the choice of excavation method. Careless blasting will result in a larger HDZ (see below) which will in turn, cause an expansion of the EDZ as compared with that developed by controlled blasting or mechanical excavation.

The HDZ is a zone where macro-scale fracturing may occur through interconnection of stress induced fractures, interaction with existing structure or through primary fractures.
developed from the excavation boundary. The effective HDZ permeability or transmissivity is dominated by the interconnected fracture system and may be significantly greater than the undisturbed rock mass (rock mass permeabilities increase by a factor of 1000+ (ANDRA 2005) times the background value). In this zone, new fractures and bedding slip are continuous at the excavation scale, and are identified by yield with significant plastic strain, fracture dilation, tensile failure, joint aperture increase and significant drop in internal stresses. The boundary with the EDZ will be sharp in brittle rocks and more gradational in ductile rocks. In spalling rocks the fracture aperture is driven by extensile separation of the fracture walls while in more ductile rocks, dilatant shearing opens up shear fractures. Where joints are present, a combination of extension and shear may be active.

The HDZ is a fractured zone that will be induced by the presence of the excavation and/or by the excavation method. Careful blasting is always required to minimize HDZ although some HDZ may be present after excavation using any technique. Support of HDZ is essential to minimize expansion of both HDZ and EDZ. Loss of HDZ material through support failure will lead to EDZ expansion. For secondary expansion of the excavation (shaft widening, cut-off construction etc.) it is possible that long term relaxation within the HDZ and EDZ will reduce the likelihood of further HDZ expansion during this subsequent construction phase.

For contracting purposes, it may be desirable to introduce a new term - CDZ or Construction Damage Zone. In this zone the fractures are entirely the result of construction method (shock, blast gases, poor blasting or poor sequencing in general). Unlike the HDZ, this zone is unlikely to expand with careful cut-off or remediation excavation (reaming or slashing of the HDZ).
Cut-off strategies should be aimed at minimizing flow along the HDZ and keying into the EDZ. The depth into the EDZ will be site dependent and will depend on the size of the EDZ. Ultimately the depth will depend on the interconnectedness of the fracture network and required reduction in the flow along the excavation boundary stipulated by the licencing authority.

1.6 Significances of EDZs
The primary concern for nuclear waste disposal underground, in relation to the EDZs, is the potential flow pathway along the damage zone which parallels the axis of the excavations. Measurement of the change in the hydraulic properties with distance away from the excavation surface is typically a required step in the characterization of the EDZs. In order to conduct advanced numerical analysis it is necessary to also understand the likely fracture network geometry, how the fractures are connected, what their orientations are and the continuity parallel to the excavation axis. With a conceptual understanding of how the damage process around underground excavations develops and the resulting typical fracture pattern, it is possible to conduct a preliminary evaluation of potential sites without having any existing underground excavations for observation. It also becomes possible to infer from continuum based models what the likely fracture pattern will be like based on the numerically delineated damage zones. This section will address the current state of understanding of how the fracture network develops in sedimentary rocks, which will be utilized in developing a continuum based approach to defining the EDZs.

1.6.1 Hydraulic Properties
Measurement of the hydraulic properties of the EDZs has been a standard means of evaluating the depths of each zone away from the excavation surface. This typically requires drilling boreholes at various locations to conduct tests at various depths along the borehole. Various other methods
(down borehole and excavation surface) have also been used. Several of these are discussed briefly to help illustrate the change in the hydraulic properties around underground excavations. These measurements result in bulk permeabilities, which do not necessarily reflect the transmissivity of individual fractures.

For transient or steady state drainage, the variation of bulk permeability or conductivity within the rock mass and the decreasing distance to the tunnel boundary must be considered. Walsh (1965), Bandis et al. (1983), and Brace (1980) recognized that the permeability of a rock mass was controlled by the geometry of discontinuities, their interconnectivity, and the size and shape of voids (pore space). A simple but widely accepted approximation for the relationship between conductivity and fracture aperture is the so-called “cubic-law” (Equation 1.1);

$$\frac{Q}{\Delta h} = C(2b)^3$$  \hspace{1cm} (1.1)

where $Q$ is the flow rate, $\Delta h$ is the difference in hydraulic head, $C$ is a constant that depends on the flow geometry and fluid properties, and $2b$ is the fracture aperture (Witherspoon et al. 1980). Volumetric strain in a fractured rock mass near an excavation boundary is primarily the result of the expansion of the fractures perpendicular to their orientation. At the excavation boundary this expansion is not necessarily uniform in all three dimensions. Therefore, $b$ in Equation 1.1 can be replaced with volumetric strain and the cubic relationship still holds true, albeit with appropriate geometric factors.

Investigations into changes in transmissivity and permeability around underground excavations have been reported by many authors (Kelsall et al 1984, Martino and Chandler 2004, Zhang et al. 2007, Baechler et al 2011, for example). Martino and Chandler (2004) compiled different measurement methods from a borehole at Canada’s URL. They demonstrated that these measurements could be used to indicate changes in transmissivity extending away from the
excavation, as shown in Figure 1.11. The excavation damage zones have been indicated at major changes in the slope of the graph in Figure 1.11 by the author.

The scope of this thesis is to understand how the fracture network develops around underground excavations in mud rocks and carbonates through laboratory testing and field observations. With this understanding in mind numerical based indicators for the dimensions of the EDZs are investigated.

Figure 1.11: Changes in the velocity (MVP lines) and transmissivity (SEPPI points) with distance from the excavation face. The HDZ, EDZ, and EIZ are delineated at major changes in the slope of the graph (modified from Martino & Chandler 2004).
1.7 Thesis Objectives

Utilizing field and laboratory properties, excavation observations and numerical modeling a comprehensive understanding of excavation damage around excavations in sedimentary rocks is presented in this thesis. This understanding is applied to continuum based numerical approaches to delineate the dimensions of excavation damage zones.

1.7.1 Part I

The primary objectives of Part I of this research are:

- To assess the intact rock properties of sedimentary rocks in Ontario to determine the range of key properties used as inputs into numerical models and to evaluate these properties on a project and formation scale.
- To develop a geotechnical classification system to associate geological naming conventions with geotechnical behaviour, particularly for mud rocks and carbonates.
- To examine the methods of measuring and estimating tensile strength of intact rock and to determine appropriate methods of calculating the direct tensile strength from indirect measurement and estimates.

1.7.2 Part II

The primary objectives of Part II of this research are:

- To assess and document excavation behaviour in mud rocks and carbonates, including:
  - New insights into the excavation behaviour of the Queenston Formation from the Niagara Tunnel Project, Niagara Falls, Canada,
  - And the excavation behaviour of a lime mudstone, the Quintner limestone, from excavations near Sargans, Switzerland.
- To understand the influence of sedimentary features on excavation behaviour, particularly focused on bedding anisotropy and bedding plane weaknesses.
To develop a comprehensive understanding of the development of EDZs in sedimentary rocks.

1.7.3 Part III

The primary objectives of Part III of this research are:

- To assess the ability of continuum based numerical modeling to delineate the dimensions of the EDZs, with specific aims to:
  - To evaluate numerical indicators which are most suitable to delineate the specific damage zones and
  - To develop a standard set of criteria to define the damage zones.
- To evaluate the sensitivity of the continuum methods to the rock mass input properties, particularly tensile strength, and stress levels.
- To evaluate the influence of anisotropy orientation on the development of excavation damage through numerical back analysis of the behaviour of the Niagara Tunnel and use these findings to forward predict the dimensions of the excavation damage zones around a shaft in the Queenston Formation.

1.8 Synopsis of Findings

The primary research objectives have been achieved with the following results, which have been summarized below.

1.8.1 Part I

In Part I of this thesis the main objective was to understand the development of damage in sedimentary rocks at the laboratory scale. The research began by evaluating an extensive data base of sedimentary rock properties from formations in Ontario. The most well represented formations in the data base were the Georgian Bay, the Queenston, the Lindsay/Cobourg formations and the Black River Group, with over 100 Unconfined Compressive Strength tests in each. These formations comprise mud rocks and carbonates. Coarse grained siliciclastic rocks
(sandstones) are relatively thin compared to other rock types in Ontario. The frequency of testing results is predominately controlled by the relationship between the near surface exposures of these formations to highly populated areas of Ontario. It was found that the Queenston and Lindsay/Cobourg rock properties were consistently represented at different localities when classified using the combined Dunham (1962) and Lungard and Samuels (1980) systems, which have been presented in this research. By understanding the potential range and average properties of the sedimentary rock properties of Ontario better feasibility and preliminary design analysis can be conducted. This is particularly important for the siting of high level nuclear waste repository sits being conducted currently in Canada.

One of the areas where there was significant less data available was tensile strength values. It can be challenging to conduct direct tensile strength testing due to sample preparation difficulties and the frequency of invalid test results. The direct tensile test does however give the true tensile strength of intact rock. Alternative indirect tests and estimates of tensile strength were compared with direct tensile test results to see if consistent relationships exist. It was determined that the BTS obtained in standard testing is generally less than the DTS and that this relationship is rock type dependent. Crack Initiation yields a reasonable estimate of tensile strength and this correlation is improved when the BTS values are reduced to DTS values by rock type specific correlations. The factor \( f \), in \( \text{DTS} = f \times \text{BTS} \), can be considered to be approximately 0.9 for metamorphic, 0.8 for igneous and 0.7 for sedimentary rocks.

The findings of the laboratory scale testing were applied to the remaining aspects of the research and were specifically used, in some cases, as numerical inputs. This aided in understanding the excavation behaviour investigations by providing an understanding of the range of sedimentary rock properties and how these properties influence excavation behaviour.
1.8.2 Part II

The main objective of Part II of this research was to understanding the excavation damage development in mud rocks and carbonates. The behaviour of the Queenston Formation from the Niagara Tunnel Project (NTP) was re-evaluated, further to the work of Perras (2009), with attention to the damage development process. It was found that the bedding planes represent a plane of weakness and a controlling factor in the damage development and ultimately the failure mechanism. Crack propagation follows the path of least resistance and therefore the fracture network tends to be characterized by fractures parallel to bedding with associated fractures migrating perpendicular to the bedding planes to reach the path of least resistance.

The Quintner limestone from Switzerland was evaluated for several reasons; the strength is similar to the Cobourg limestone – the proposed host rock for the DGR, the depth of some of the observed excavations are similar to the proposed DGR, a variety of failure modes occur including spalling. Different failure modes were observed to occur in the limestone depending on the depth below the ground surface. Brittle spalling was observed to begin in areas where the excavation geometry created a stress concentration at around 550 m deep, with an estimated 0.8 - 0.9 Ko ratio. Calcite veins were found to influence the crack propagation directions, similar to the bedding planes in the Queenston.

The conceptual stages of the damage development around underground excavations in horizontally bedded sedimentary rocks with high horizontal stresses are presented in Part I. Conceptual fracture networks in horizontally bedded sedimentary rocks with different orientations of the maximum stress are also presented. The laboratory and field work from this thesis have been used to develop a conceptual the understanding of excavation damage in
sedimentary rocks. This has been applied to the development of the numerical research in the final section of this thesis.

1.8.3 Part III

In Part III of this thesis the main objective was to determine appropriate numerical indicators that could be used to delineate the dimensions of the excavation damage zones. This was achieved by simulating excavations using the Damage Initiation and Spalling Limit approach developed by Diederichs (2003, 2007).

Using three data sets, a granite, a limestone and a shale, as well as two nominal stress regimes per rock type, the effect of the variability in the input properties on the numerical dimensions of the EDZs was explored. It was demonstrated that both an inner and outer excavation damage zone (EDZi and EDZo) could be differentiated using the reversal point in the volumetric strain (contraction to extension). This indicates the transition between a confined micro-damaged state (EDZo) and a potentially dilated EDZi. The outer boundary of the highly damaged zone, HDZ, is related to volumetric strain and a reduction in minor stress confinement. Guidelines are suggested for determining the dimensions of the EDZs around circular excavations. Based on a statistical evaluation of the numerical results it was determined that the mean damage zone dimensions could be predicted using the following equations:

\[
\text{EDZo} / a = 1 + 0.6 (\pm 0.07) (\sigma_{\text{max}}/\text{CI} - 1)^{0.6(\pm 0.04)} \tag{1.2}
\]

\[
\text{EDZi} / a = 1 + 0.4 (\pm 0.07) (\sigma_{\text{max}}/\text{CI} - 1)^{0.5(\pm 0.07)} \tag{1.3}
\]

\[
\text{HDZ} / a = 1 + 0.2 (\pm 0.06) (\sigma_{\text{max}}/\text{CI} - 1)^{0.7(\pm 0.25)} \tag{1.4}
\]

where \(a\) is the excavation radius, \(\sigma_{\text{max}}\) is the maximum tangential stress at the excavation boundary and \(\text{CI}\) is the crack initiation threshold.
The sensitivity of the numerically predicted dimensions of the EDZs was evaluated by looking at different rock properties (CI and tensile strength), different stress magnitudes and different numerical mesh arrangements. It was found that the dimensions were most sensitive to the tensile strength used as an input into the model.

Three modelling approaches were used to back analyse the brittle failure process at the NTP: the Damage Initiation and Spalling Limit approach, the laminated anisotropy modelling approach, and a ubiquitous joint approach. Analyses were conducted for three tunnel chainages: 3000, 3250, and 3500 m. All approaches could produce a notch of similar geometries to that measured at each chainage analysed. The Laminated Anisotropy Method (LAM) approach was the most consistent at accurately capturing chord closure measurements. For the laminations, good agreement with the chord closure was achieved with joint normal to shear stiffness ratio of 1 to 2. This understanding was applied to a shaft excavation in the Queenston Formation at the proposed DGR site for low and intermediate level nuclear waste storage near Kincardine, Ontario. The maximum damage depth was 1.9 m; with an average of 1.0 m. The models show that the observed normalized depth of failure at the NTP would over-predict the depth of damage expected in the Queenston Formation at the DGR.

A discussion of the above findings centered around the influence on cut-off structures, designed to intersect the damage zones, is presented and future areas of research for cut-off design strategies are suggested. The specific aspects of each chapter are briefly outlined in the next section.

1.9 Thesis Outline
This thesis is structured as a manuscript based thesis. The majority of the chapters are or will be submitted to journal publications. Therefore each chapter has an introduction with relevant
review material, followed by new findings and detailed discussion. The exception to this is Chapter Two which is a review of the geological history of sedimentary deposition in Ontario and a review geological classification systems for sedimentary rocks.

The main part of the thesis is divided into three parts; Part I documents the understanding of the mechanical properties of sedimentary rocks with a focus on mud rocks and carbonates, Part II documents the understanding observations of excavation behaviour in sedimentary rocks with a focus on mud rocks and carbonates, and Part III documents numerical methods of predicting the dimensions of the different damage zones using continuum models and the sensitivity of the dimensions to the numerical inputs. The specifics of each Chapter are summarized below.

1.9.1 Part I: Understanding the Mechanical Properties of Mudrocks and Carbonates
Chapter Two presents a review of the geological setting of Ontario and geological classification systems. This information is critical for the sedimentary rock properties of Ontario and to ensure that rock properties from only similar rocks, as classified, are used correctly in this thesis.
Chapter Three presents a collection of sedimentary rock properties, field and laboratory, from the literature and laboratory test results presented as part of this study. The variability of these properties is examined to understand which sedimentary formations in Ontario have the most consistent properties. Understanding the potential range of properties can be utilized for preliminary design purposes or to identify potential host rock formations for underground construction. By understanding the properties at the laboratory scale predictions can be made for rock mass properties and therefore underground excavation behaviour. Chapters Two and Three will be combined into a single submission for publication.

The tensile strength of rocks was explored in more detail in Chapter Four to determine which method of tensile strength determination most closely reflects the direct tensile strength.
Most indirect methods of determining tensile strength were found to overestimate the direct tensile strength and reduction factors are suggested. This chapter has been accepted for publication and appears in the form submitted to the Geotechnical and Geological Engineering Journal.

1.9.2 Part II: Excavation Damage Zone Characteristics in Mudrocks and Carbonates

Chapter Five summarizes observations from the Niagara Tunnel Project, a 14.4 m diameter water diversion tunnel in Niagara Falls, Canada, with emphasis and new insights into the behaviour of the Queenston Formation. The Queenston Formation ranges from a claystone to a siltstone with the majority of the samples tested being mudstone. This example represents one of the best examples of mudrock behaviour in Ontario. The Queenston Formation is present near the surface and within the subsurface of most of Ontario. For underground storage of nuclear waste in Ontario access shafts will have to pass through the Queenston Formation as it represents an isolating layer in the sedimentary sequence of Ontario. This chapter has been submitted as it appears to the Bulletin of Engineering Geology and the Environment, the official journal of International Association for Engineering Geology and the Environment (IAEG).

Chapter Six summarizes carbonate rock excavation behaviour from Ontario and documents field work conducted in Switzerland of excavations in the Quintner lime mudstone. The majority of excavations within limestone in the Michigan and Appalachian sedimentary basins are relatively shallow (<300 m) compared to the depth of the proposed nuclear waste repository for low and intermediate level nuclear waste in the Cobourg limestone. To better understand the variety of excavation behaviours in limestone field work was conducted at the Gonzen mine at depths between 600-700 m within the Quintner limestone. This work was
supplemented with a review of other excavations within the Quintner limestone and is in preparation for publication.

In Chapter Seven, based on the laboratory understanding of crack initiation and propagation from Chapter Three and Four, along with the observations from Chapters Five and Six, a comprehensive understanding of the development of excavation damage in sedimentary rocks is presented. Conceptual damage zones and fracture networks are summarized at the end of Part II of the thesis. Chapter Seven is in preparation for publication.

1.9.3 Part III: Predicting the Dimensions of Excavation Damage Zones

In Chapter Eight the use of continuum numerical models to predict the dimensions of the Excavation Damage Zones is studied. Numerical indicators to determine the dimensions are defined and applied to three rock types; granite, shale, and limestone to understand the differences in the numerical behaviour. The dimension sensitivity to a variety of numerical factors is examined. Tensile strength as an input property was found to have a significant influence on the dimensions. This chapter has been submitted as it appears to the International Journal of Rock Mechanics and Mining Sciences.

Chapter Nine examines the role of anisotropy on damage development in the Queenston Formation, back analyzing the behaviour from the Niagara Tunnel Project. The orientation of the excavation relative to the plane of anisotropy is examined and forward damage dimension predictions for a shaft in the Queenston are presented. This chapter will be part of a special publication of the Rock Mechanics and Rock Engineering Journal based on the workshop on Failure Prediction in Geotechnics at the Salzburg Congress.

Discussion of the main findings of this research are presented in Chapter Ten with an emphasis on the impact to mitigation strategies for reducing the flow along the EDZ. Cut-offs
constructed to intersect the EDZ perpendicular to the excavation axis are the most common mitigation method and the influence on the expansion of the EDZ during construction of simple cut-off geometries is examined for discussion purposes only.

The limitations of this study are identified and future research is suggested. Since the majority of the chapters in this thesis are or will be submitted for publication in international journals the references are shown at the end of each chapter.

1.10 References


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PART I: Understanding the Mechanical Properties of Mudrocks and Carbonates
Chapter 2: A Review of the Sedimentary Geology of Ontario and
Geological Classification of Sedimentary Rocks

2.1 Introduction
A number of notable underground excavations in Ontario have either been completed, are under
construction or have been proposed in sedimentary rock formations across the province. These
include the Niagara Tunnel Project (NTP), the Ottawa Light Rail Transit (OLRT) Project, and the
Deep Geological Repository (DGR) for Low and Intermediate Level Nuclear Waste. The location
of these projects are shown in Figure 2.1. The NTP is a hydropower diversion tunnel which was
constructed with a tunnel boring machine in the Appalachian sedimentary basin in the city of
Niagara Falls, Ontario. The OLRT project is currently under construction in the sedimentary
rocks of the Ottawa Embayment. This project is intended to increase the capacity of the public
transit system in the downtown core of Ottawa, Ontario. The DGR for Low and Intermediate
Level Nuclear Waste is proposed to be constructed near the town of Kincardine, Ontario in
sedimentary rocks of the Michigan Basin. Data from these projects are used throughout this
thesis; to examine the sedimentary rock properties of Ontario (Chapter 3), to understand the
failure mechanisms at the NTP (Chapter 5), to back analyse the behaviour from the NTP and to
forward predict the behaviour and damage zone dimensions of the DGR (Chapter 9).

This chapter reviews the sedimentary geology of Ontario to provide context for
subsequent chapters, which discuss the mechanical properties, excavation behaviour, and
projected behaviour of the different sedimentary rocks from across Ontario. The tectonic history
and sedimentary basin development is reviewed in the context of the formations for which

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1 This chapter, in combination with Chapter 3, is in preparation for submission to an international journal.
mechanical testing data is available in the literature or presented as part of this study. Preparing this review of the geology and during the collection of the mechanical testing data for this research project brought to light a need to better communicate the use of geological classification systems to the engineering geology community. The geological classification systems are reviewed in this chapter and utilized throughout the thesis to ensure consistent use of the terminology when describing sedimentary rocks.

Figure 2.1: Regional geological setting of the Southern Ontario highlighting the sedimentary rocks of interest in this study and the key projects; the Niagara Tunnel Project (NTP), the Ottawa Light Rail Transit (OLRT) Project and the Deep Geological Repository (DGR). Depth contours are in meters above sea level (modified from Mazurek 2004).
2.2 Paleozoic Geology of Southern and Eastern Ontario

The sedimentary rocks of Southern and Eastern Ontario overlie granitic and metamorphic rocks of the Precambrian basement or the Canadian Shield, as indicated in Figure 2.1. These sedimentary rocks were deposited from Cambrian to Mississippian time into three major depositional centres; the Michigan Basin, the Appalachian Basin, and the Ottawa Embayment. The relevant tectonic history during and following deposition, the history of deposition and the formation characteristics are reviewed with a specific focus on the descriptions of the formations for which geomechanical testing data was available at the time this project was completed.

The sedimentary rocks of Ontario have been deposited in basins. Sedimentary basins are depressions in the Earth’s crust that allow for the accumulation of sediments and are typically of tectonic origin. Tectonic forces create relief, which allows for weathering and erosion of the higher ground and the sediment created by these processes to accumulate in the lower areas.

In Southern Ontario, the Appalachian and Michigan basins are separated by a Precambrian basement high called the Algonquin and Findlay Arches, which run in roughly a SW-NE direction, as shown in Figure 2.1. Similarly, the Ottawa Embayment was separated from the Appalachian basin by the Frontenac Arch and Adirondack Mountains. These features acted as structural and topographic controls during deposition. The Adirondacks protected the sedimentary rocks of the Ottawa Embayment from Paleozoic folding (Bally 1989) as indicated by the limit of the Appalachian Front in Figure 2.1. The tectonic history during and following deposition will be briefly reviewed in the following sub-section.

2.2.1 Tectonic History of the Sedimentary Basins of Ontario

The three sedimentary basins of Ontario have developed in large part due to mountain building processes on what is now the east coast of North America. The Appalachian Basin is a foreland
basin that was created during the Taconian orogen. Bally (1989) suggests that such foreland basins are associated with fold belts, and to support this structure, the basin is elongated parallel to the Appalachian front, with a thickness of up to 12 km (Johnson et al. 1992). The Michigan Basin is an intracratonic basin that is circular in shape with a diameter of 500 – 600 km and a depth of up to 4 km (Johnson et al. 1992). The Ottawa Embayment is an elongate shaped basin which reflects, in part, deposition into a sedimentary basin bordered by Precambrian highlands to the north, west, and south (Figure 2.1). The elongate shape also reflects the fault-block fabric of the Ottawa-Bonnechere Graben which runs parallel to the long axes of the Embayment (Dix and Robinson 2003). The sedimentary sequence present today is in the order of 200 m.

The tectonic history has played an important role in shaping and modifying the geological environment during deposition. Precambrian to Cambrian rifting marked the beginning of an initial episode of subsidence and deposition within the Michigan Basin (Sanford et al. 1985). Differential erosion and variable Precambrian topography resulted in uneven deposition of Cambrian sediments. This cycle ended in the early Middle Ordovician during regional uplift. Middle Ordovician to Mississippian sedimentary formations reflect the complex interaction between regional-scale tectonic forces, sedimentation, and eustatic sea level fluctuations associated with the Appalachian-Caledonian orogeny (Johnson et al 1992, Sanford 1993).

Three pulses of tectonic activity have affected the sedimentary formation of Ontario; including the Taconic (Ordovician), Caledonian/Acadian (Silurian-Devonian) and Alleghenian (Carboniferous-Permain) (Sanford et al. 1985). During the deposition of sediments in the basins, the Algonquin arch is believed to have been actively alternating between subsiding and uplifting which periodically connected and disconnected, respectively, the two basins. This movement resulted in sedimentary formations which thin over the arches or are truncated (Stearn et al. 1987).
1979). Multiple unconformities have been reported by Cercone and Pollack (1991), indicating intermittent up-lift of these arches during deposition.

Middle Ordovician deposition occurred on a south-eastward-facing shelf and ramp that was laterally extensive across Ontario in both the Appalachian and Michigan Basins, as shown in Figure 2.2a (Melchin et al. 1994). This expansive carbonate system could be formed, because during deposition, the Algonquin and Frontenac Arches had subdued relief because of large-scale eastward-tilting of the Laurentian margin. The termination of Taconian movements created an unconformity at the Ordovician-Silurian boundary.

Deposition was rejuvenated during the Acadian Orogeny (Late Silurian) which was driven by the collision of the North American and African plates. This collision re-activated the Algonquin-Findlay arch system and returned the Michigan Basin to a more isolate (circular shaped) depositional centre. This created contrasting depositional environments between the Michigan and Appalachian Basins with carbonate deposition being restricted to the central area of the Michigan basin (Figure 2.2b) and more clastic sediment input in the Appalachian. Continued erosion of the highlands deposited clastic sediments during the Silurian, which choked the carbonate formation during the early Acadian (Bally 1989). Foreland loading and tectonism renewed platform margin subsidence allowing for marine sediments to again be widely distributed across the arch system in the mid to late Devonian (Armstrong and Carter 2006).

The Alleghanian Orogeny was the last active tectonic activity to result in sediment deposition. The sedimentary formations have thick coal deposits in the Appalachian foreland and in the central area of the Michigan Basin. In Ontario, however, there is a lack of sediment deposition during this time over the arches. More specific depositional characteristics of the formations will be described in the next section.
Figure 2.2: Ordovician depositional facies showing transition from a) carbonate shelf and ramp dominated to b) clastic dominated due to the Taconic Orogeny (from NWMO 2011 after Sanford 1993).

2.2.2 Depositional History
The sediments deposited in the basins of Ontario range from Cambrian to Mississippian, as shown in Figure 2.3. In general, the bedrock strata dip shallowly at 3 to 6 m/km south-westward.
along the crest of the Algonquin Arch (Figure 2.4) and 3.5 to 12 m/km perpendicular to the strike of the arch into the basins as shown in Figure 2.1 (Armstrong and Carter 2010). In the Ottawa Embayment the bedrock also dips at a shallow angle, as shown in Figure 2.4.

The stratigraphy of the three main sedimentary depositional centres of Southern and Eastern Ontario are shown in Figure 2.5 and the formations for which rock mechanical testing has been reported in the literature or included as part of this study are indicated. The mechanical testing data used in this thesis are summarized in Appendix A. The descriptions of the formations of interest for the rest of this research are described in the following sub-sections.

2.2.2.1 Cambrian
Thick layers of Cambrian rocks are present in some portions of Ontario and in other areas the Cambrian rocks are very thin to absent. In general, the Cambrian deposits are considered to be marine sandstones and dolostones deposited in transgressive seas (Hamblin 1999). The only mechanical testing found in the literature comes from the DGR site (Gorski et al. 2011), where on Cambrian sandstones. The Cambrian samples of interest were cream to orange brown, coarse-grained quartz sandstone (Sterling 2010).

2.2.2.2 Ordovician
Were the Cambrian rocks are missing the Middle Ordovician Shadow Lake Formation directly overlies the Precambrian basement rocks. The Shadow Lake Formation consists of red and green sandy shale and sandstone with silty dolomite. The Gull River Formation conformably overlies the Shadow Lake (Armstrong 2000) and is predominantly lime mudstone and dolomite. The Ordovician rocks are exposed at the surface in eastern and south-central Ontario and continue in the subsurface of south-western Ontario. Due to their close proximity to urban centres, these rocks are often studied for excavation projects.
Figure 2.3: Geology of Ontario, showing major groups and formations (OGS 1991).
Figure 2.4: Generalized geological cross sections (top) from Ottawa to Lake Ontario showing the Ottawa Embayment and (bottom) from Toronto to Windsor oblique to the axis of the Algonquin Arch (modified from Helmstaedt and Godin (2008) and Mazurek (2004)).
Figure 2.5: Composite stratigraphy of the Michigan, Appalachian, and Ottawa Embayment areas in Ontario, showing correlation between formations, major unconformities and highlighting formations with mechanical testing data (after Armstrong and Dodge 2007).
Following the deposition of lime muds, coarser, fossil-rich limestones of the Bobcaygeon Formation were deposited. This formation has a nodular texture in the lower part and thin shale interbeds and storm beds in the upper (Armstrong and Dodge 2007). The lower part is similar to the Coboconk Formation and the upper to the Kirkfield Formation in Southwestern Ontario (Figure 2.5). The Verulam Formation gradationally overlies the Bobcaygeon and consists of interbedded shale and limestone (Armstrong and Dodge 2007). Golder and Associates (2011) described the Verulam as fine-grained and crystalline in the Ottawa area. It is similar to the Sherman Fall Formation in South Western Ontario (Figure 2.5). The Lindsay Formation is a fossiliferous argillaceous limestone. It has a nodular texture due to the irregular argillaceous wisps formed through bioturbation and compaction during diagenesis. It is similar to the Cobourg Formation of the Michigan Basin. The Collingwood Member of the Cobourg is a black, fissile lime mudstone, and Melchin et al. (1994) indicates that this was deposited at the peak of the marine transgression in a deep-shelf environment.

The Upper Ordovician begins at the Blue Mountain Formation, which is distinctly non-calcareous (Gartner Lee 2008) at its contact with the Middle Ordovician strata. Interbeds of limestone and calcareous siltstone become more frequent as it gradationally transitions into the Georgian Bay Formation. In eastern Ontario, these two formations are equivalent to the Billings and Carlsbad formations, respectively. The Queenston Formation is Late Ordovician in age and is part of a compound deltaic, shallowing-upward sequence (Stearn et al. 1979). The formation is a red mudstone with siltstone, sandstone, and limestone interbeds (Johnson et al. 1992).

The top of the Queenston Formation was exposed to weathering and erosion, creating an unconformity. In the Ottawa Embayment, the Queenston Formation is the youngest rock formation prior to the deposition of Quaternary sediments.
2.2.2.3 Silurian

The unconformity at the top of the Queenston Formation is disconformable with the Lower Silurian Whirlpool sandstone in the Niagara Region (Perras et al. 2013). The Cataract and Clinton Groups are both Lower Silurian in age and both were deposited in deltaic and shallow marine environments (Curricie & Mackasey 1978). The Whirlpool sandstone gradationally transitions to the Manitoulin Formation, a fine-grained dolomitic limestone moving north-westward from Niagara as the Whirlpool thins and becomes discontinuous (Bergstrom et al. 2011). The Mantoulin marks the return to carbonate forming conditions, during the transgression that followed the disconformity, on a southwest dipping carbonate ramp (Armstrong and Carter 2010). With minor fluctuations in the sea level, the depositional material alternated between sand, clay, and calcareous shell fragments, forming the Cabot Head and Grimsby Formations (Winder & Sanford 1972). The Cabot Head is composed of non-calcareous shale with interbedded sandstone and limestone (Armstrong and Carter 2010). Limestone beds are absent in the Niagara Region, where the grey shales with interbedded sandstone are called the Power Glen Formation (Perras et al. 2013). The Dyer, Wingfield, and St. Edmund formations are included in the Cataract Group; however, due to their limited exposure near surface in urban areas they have been studied little.

The Clinton Group is Middle Silurian in age and is comprised of the Thorold, Neahga, Reynales, Irondequoit, and Rochester Formations, which were deposited in a shelf-edge environment (Winder & Sanford 1972). Reworked Grimsby detritus formed the Thorold Sandstone as sea levels increased. With continuing sea level changes in an intertidal and lagoonal environment, the Neahga shale and Reynales dolostone formations were deposited (Winder & Sanford 1972). These formations pinch out moving north and west (Armstrong and Dodge 2007) and correspond to the Fossil Hill Formation, which is fossiliferous crystalline dolomite. In the
Michigan Basin, this marks the youngest rocks of the Clinton Group; however, in the Appalachian Basin, the Clinton Group includes the Irondequoit, Rochester, and Decew. These correspond to the Lions Head and Wiarton formations of the Amabel Group in the Michigan Basin. The Irondequoit Formation is a crinoidal dolomite, and the Rochester Formation is a dolomitic shale (Winder & Sanford 1972). Patch reefs form the lower Lockport Formation, and regional reefs form the upper Lockport and Guelph Formations (Tesmer 1981). The Lockport has been divided into three members in the Niagara region: the Gasport, Goat Island, and Eramosa. All three members can be considered limestones, and they correspond to the Lions Head dolomite and the Wiarton limestone of the Amabel Formation.

The Upper Silurian begins partially within the Guelph Formation. The pinnacle reef belt is thickest along the crest of the Algonquin Arch and thins away from the arch in the inter-reef locations (Gartner Lee 2008). These reef complexes confine the Michigan basin and allow for evaporates, of the Salina Group, to be deposited with interbedded argillaceous dolomite and shales. The Salina units vary cyclically, grading upward from basal carbonates to anhydrites to halite, with the top of each evaporate cycle often marked by shale (Armstrong and Carter 2006). The Silurian deposits are capped with the Bass Island (South Central Ontario) and Bertie (Niagara Escarpment) formations which are both dolomite.

2.2.2.4 Devonian
The Lower Devonian formations are the Oriskany and Bois Blanc. Testing has been conducted on the Bois Blanc dolomite and limestones. The Lower to Middle Devonian transition occurs within the Detroit River Group in the Michigan Basin, which corresponds to the Onodaga Formation in the Appalachian Basin. The base of the Detroit River Group is Sylvania sandstone, which only occurs locally in the subsurface of the Windsor area (Russell 1993). This unit grades into the
fossiliferous limestones and dolomites of the Amherstburg Formation. The Lucas Formation sharply overlies the Amherstburg. The Lucas is predominately limestone and dolomite with anhydritic and sandy beds (Armstrong and Dodge 2007). The siliciclastic layers indicate the waning influence of the reef complexes. A disconformity exists between the Lucas and the overlying Dundee limestones. Above this exists the Hamilton Group, which is predominately calcareous shale. No testing results for the formations of this group have been found in the literature. The youngest (Upper Devonian) rock formation for which testing data was found was the Kettle Point Formation, an organic rich shale with interbedded siltstone layers. Younger sedimentary formations are absent for the geomechanical literature and are not discussed further in this review.

2.2.2.5 Quaternary Geology and Glaciation
In North America there have been nine glacial events during the Quaternary Period (Peltier 2011). The unconsolidated sediments consist of the following; ground moraine or glacial till, glaciofluvial deposits, glaciolacustrine deposits, and ice contact deposits formed at the margin of the glacier. The weight of the ice sheet depressed the surface of the earth by approximately 600 m (Peltier 2011). With the retreat of the glacial ice the earth began to rebound and is continuing to do so today at a rate of approximately 1.5 mm/year in the Great Lakes region (Peltier 2011). The glacial loading would increase the vertical stress on the rock mass and the pore water pressure would also increase during glaciation. The stress field would further be modified by erosion of the bedrock surface. Ranges of erosion for one single glaciation are between 10 m to 600 m locally, with the high end of the range reported to be unlikely a uniform occurrence (NWMO 2011). The glacial impact on the sedimentary rock properties and on underground excavations is beyond the scope of this research.
The sedimentary geology of Ontario has been briefly reviewed as it relates to the geomechanical study presented in Chapter 3. During this review it became evident that geological classification systems are an important aspect that allows the terminology describing sedimentary rocks to be used in a consistent manner. This become increasingly important when trying to compare geomechanical data from the same formation, but reported by different authors or from differ geographic areas. For this reason the next section will review some of the geological classification systems commonly used in geology for sedimentary rocks and discuss the systems which are employed throughout this thesis.

2.3 Geological Classification of Sedimentary Rocks

Grabau (1904) realized the large grouping of sedimentary rocks called limestones was inadequate to distinguish between the many varieties. Further development in geological classification systems for limestones by Folk (1959, 1962) and Dunham (1962) are still widely used today. Dunham’s (1962) classification system is the most widely used in geological sciences because of its ease of use in the field. Folk’s (1962) system requires thin section analysis which is not needed to utilize Dunham’s (1962) system. Embry and Klovian (1971) and James (1984) further developed Dunham’s (1962) system to help account for larger fossil fragments and different binding organisms and agents.

Similarly, siliciclastic classification systems have been developed for coarse-grained (Pettijohn et al. 1987) and fine-grained (Lungegard and Samuels 1980) rocks to distinguish between different grain types and sizes.

Geotechnically, the rock in consideration for a project is classified according to a rock mass scheme, such as the Rock Mass Rating (RMR) system of Bieniawski (1973, 1989), the Q system of Barton et al. (1974), or the Geological Strength Index (GSI) of Hoek (1994) and Hoek
et al. (1995). The GSI system was developed to reduce intact rock properties based on geological conditions and was largely motivated by the increasing use of continuum codes for design.

The use of geological systems to classify rock masses is not widespread in the geological engineering community. This may be because standard compressive testing typically yields a wide range of values for all rock types and as such the need to further classify each individual sample seems irrelevant. In addition, average values for standard mechanical properties, such as Young’s Modulus ($E_i$) or Unconfined Compressive Strength (UCS), are typically used, and variations may be acceptably captured by considering the range for design. However, at the grain scale, there can be significant changes, for example in the limestone characteristics, which can influence the micro-mechanical behaviour, particularly the fracture initiation and propagation thresholds. By classifying each sample tested this influence maybe more easily understood.

2.3.1 Carbonate Classification

Folk (1959, 1962) proposed a method of classifying carbonate rocks based on petrographic thin section analysis. This system is based on two principal components, matrix and allochems (grains), as shown in Figure 2.6. A distinction is made between four different types of allochems, as well as those rocks lacking allochems or reef builders. In this system, the matrix material, either sparry or micrite calcite cement, is used as a major division. The definitions given by Folk (1962) are as follows:

- **Sparry**: coarse calcite cement; clear crystals $>10$ μm
- **Micrite**: very fine, sub-translucent crystals 1–4 μm, also called microcrystalline cement

These terms are conjugated with the type of allochems present, and the terminology can get confusing. The percentage of the matrix within a sample and the depositional environment can be inferred from the naming convention, as shown in Figure 2.6.
Figure 2.6: Folk’s (1959, 1962) textural classification of carbonate rocks.
Dunham’s (1962) system, outlined in Figure 2.7, addresses the issue of depositional environment by considering the allochem-matrix relationship, in terms of percentage volume or packing arrangement. The following class definitions, as outlined by Jones and Desrochers (1992), are considered in this paper, unless otherwise indicated:

- **Mudstone**: fine-grained; < 20 μm (Dunham 1962); originating from ooze, disintegration of fossils or grains, or possibly from inorganic precipitation; associated with low energy or binding organisms in medium-high energy
- **Wackestone**: transitional; matrix supported; 10% < volume grains < 20-30%
- **Packstone**: transitional sediment; grain support; volume of grains 20–30%
- **Grainstone**: high energy; no mud; typical of shoals, and beaches
- **Dolomite**: 1:1 ratio of Ca\(^{2+}\) to Mg\(^{2+}\) ions; predominately CaMg(CO\(_3\))\(_2\); considered to be crystalline dolomite in this thesis
- **Crystalline**: recrystallization of limestone; change in grain/crystal size without a change in mineralogy; considered to be crystalline limestone in this thesis

Both dolomitization and recrystallization are gradational processes and affect the structure of the rock mass at the micro scale. In Dunham’s (1962) system, both dolomite and crystalline limestone can be classified under *Crystalline*.

In Dunham’s (1962) classification system, the naming convention involves selecting the appropriate root word and adding a prefix to describe the most appropriate allochem(s) or other features. For instance, a packstone is grain supported with a fine carbonate mud matrix. An appropriate prefix might be oolithic, which would imply that the rock is composed of ooids. This in turn implies that the environment of deposition was near the carbonate factory, where the grains were being produced, but sufficiently far that the energy of the environment did not
remove the fine-grained mud that was deposited along with the grains. Dunham (1962) also used terminology from siliciclastic sedimentology, which geological engineers are generally familiar with. This means that the descriptive terminology of Dunham’s system is more easily understood than that of Folk (1959, 1962).

Dunham’s (1962) system is adopted for its ease of use in the field and for the more commonly used terminology. In this study, the characteristics that contribute to fracture initiation and propagation in mudstones, wackestones, packstones, and crystalline limestone are examined.

![Dunham's (1962) depositional classification of carbonate rocks](image.png)

*Figure 2.7: Dunham’s (1962) depositional classification of carbonate rocks (from Loucks et al. 2003).*
These carbonate type names are typically used in construction projects worldwide. Other types are less commonly utilized in the construction industry.

The depositional environment controls the type of carbonate rock formation. The water temperature, depth, salinity, and siliciclastic input are important factors, and James and Choquette (1990) state that to “understand limestones for any purpose it is imperative to decipher the often complex series of processes that have modified their texture and composition”. After deposition, the main processes of diagenesis include compaction, cementation, dissolution, and recrystallization. These processes also influence the micro-mechanical behaviour of carbonates.

2.3.2 Siliciclastic Classification

Lundegard and Samuels (1980) developed a system for classifying fine-grained rocks for field use. They state that a distinction between fine-grained rocks is needed, and they proposed subdivisions based on grain size and induration, as shown in Figure 2.8. They define these fine-grained rocks as mudrocks, which includes the sub-group shale (mud or clay). The term shale is often used in geotechnical literature to describe all fine-grained sedimentary rocks that are not sandstones or carbonates. The distinction between shales and siltstones may prove to be useful when describing their mechanical behaviour.

The most common classification of sandstones is derived from the original work of Dott (1964). Williams et al. (1982) modified the original work such that mudrocks begin when there is 50% matrix material, see Figure 2.9.

2.3.3 Mixed-Sediment Classification

Siliciclastic input inhibits the growth of carbonate producing organisms (Mount 1985). However, siliciclastic material is mixed with carbonate material, and there have been attempts to develop mixed sediment classification systems.
Table 2.8: Mudrock classification based on induration and silt content with description of lamination varieties identifiable in the field (modified from Lundegard and Samuels 1980).

<table>
<thead>
<tr>
<th>Indurated</th>
<th>Lamination Varieties</th>
</tr>
</thead>
<tbody>
<tr>
<td>Laminated</td>
<td>Siltstone</td>
</tr>
<tr>
<td>Non-laminated</td>
<td>Mudstone</td>
</tr>
<tr>
<td>Silt Fraction</td>
<td>2/3</td>
</tr>
<tr>
<td>1/3</td>
<td></td>
</tr>
<tr>
<td>Clayshale</td>
<td>Claystone</td>
</tr>
</tbody>
</table>

Figure 2.9: Classification of coarse-grained siliciclastic sedimentary rocks by composition (from Pettijohn et al. 1987).
Mixing can be observed as interbedded layers or as disseminated grains. The latter can be indicative of depositional environment changes or sporadic input from siliciclastic sources, such as windblown sands. Some organisms, such as sponges, build quartz into their skeletal structure, which leave siliceous skeletal fragments called spicules in the rock record (Uriz et al. 2003). 50% carbonate is used as the cut-off point in differentiating between siliciclastic and carbonate rocks.

Mount (1985) proposed a mixed siliciclastic-carbonate classification system based on texture and composition. At its simplest level, the system divides material into four components: siliciclastic sand or mud and carbonate sand or mud. The system is complicated, although Mount (1985) does provide the framework for understanding the relationship between siliciclastic and carbonate materials.

In Mount’s (1985) classification system the transition from carbonate to siliciclastic material occurs largely at fine grained boundaries (mud), as highlighted in Figure 2.10. There is a small window of transition for larger grains (sand sized). Since carbonate growth is inhibited by siliciclastic input, it is most likely that the input energy would decrease first before deposition of siliciclastic grains. The exception could be landslides which could deposit coarse grained material on top of carbonate rocks.

Folk (1954) states that for classification systems to be useful they need to be straightforward to use and include enough detail to make significant distinctions between each class. With this in mind and following the understanding that the transition from siliciclastic to carbonate typically occurs at mud sized grains, a combined siliciclastic-carbonate classification system is proposed in Figure 2.11. This system combines that of Lundegard and Samuels (1980) for mudrocks with Dunham’s (1962) carbonate classification system. Sandstones are left unclassified. The grain size ranges are based on the Udden-Wentworth scale (Wentworth 1922).
The combined classification system (Figure 2.11) will be applied to a data set of engineering properties of sedimentary rocks from Ontario in Chapter 3 to distinguish between different geomechanical behaviours.

Figure 2.10: Mount’s (1985) mixed siliciclastic and carbonate classification system.
Figure 2.11: Geological classification system for sedimentary rocks. Rock types based on Lundegurd and Samuels (1980) and Dunham (1962) with grain size ranges based on the Udden-Wentworth scale (Wentworth 1922).
2.3.4 Summary

The sedimentary formation in the Appalachian, Michigan, and Ottawa Embayment basins cover a large part of Southern Ontario. Deposition began in the Cambrian period, and in Ontario, these sediments were largely sandstones derived from the Precambrian shield. The shallow shelf environment allowed development of carbonates, and these areas were bordered by steeper slopes where shale could accumulate during the Lower Ordovician. The Taconic orogeny, which began in the Middle Ordovician, reversed the direction of sedimentation by uplifting mountains along the eastern margin of the craton, which were in turn eroded and deposited into the forming basins. The siliciclastics spread westward becoming interbedded with and displacing the limestones.

The Silurian period marks the decline in sediment input from the east as the highland areas were eroded. This allowed for deposition of carbonate rocks once again, starting in the western parts of the basins and moving in an easterly direction (Stern 1979). Uplift as part of the Acadian Orogeny once again displaced the limestone factory during the Devonian period. These cyclic changes in deposition, as well as local variations in the depositional environment, lead to interbedded sedimentary units.

Although thick sequences of similar geological units exist, small variations can influence the mechanical strength and stiffness of rock. Appropriate geological classification of different units and formations, using the systems discussed in this chapter, is standard practice.

These systems help identify the characteristics of the rock mass and are largely based on grain size and textural composition. Owing to the variable nature of sedimentary deposition, the classification at one site does not necessary apply to another, even within the same formation. Hence, each sample should be classified individually before further geological or geotechnical analyses are conducted on the sample.
2.4 References


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Chapter 3: Engineering Properties of Sedimentary Rocks in Ontario

3.1 Introduction

Intact rock properties from a variety of formations across Ontario, obtained from both literature and from the current research, were used to examine the relationships between field and laboratory values. The majority of data from the literature have been obtained from construction projects in cities such as Toronto or Ottawa. Additional test results from boreholes drilled for investigation at the Niagara Tunnel Project, the Darlington Nuclear Generation Station, and the proposed Deep Geological Repository, are also presented.

The location of the three main projects, for which new test data is presented, are shown in Figure 3.1 along with the sample locations from the literature which support this analysis. The Niagara Tunnel Project is a 14.4 m diameter water diversion tunnel that was excavated using a Tunnel Boring Machine under the city of Niagara Falls, Canada. The 10.2 km of tunnel passed through 11 formations of the Niagara Region. Extensive investigation drilling and mechanical testing were conducted. The Darlington Nuclear Generation Station is located on the northern shore of Lake Ontario, east of Toronto, Ontario. Investigation drilling for the station foundations, water cooling and discharge tunnels were conducted prior to construction. Additional testing from core samples from these two sites were tested by the author, at the Royal Military College (RMC) of Canada, as part of this research. A proposed Deep Geological Repository for Low and Intermediate Level Waste (I&ILW) is currently under review for construction licencing in the Bruce Peninsula near the town of Kincardine, Ontario. Eight boreholes have been drilled from surface, two of which went down to the Cambrian sediments which overlie the Precambrian basement. Recent testing results, not yet published, have been incorporated into this research courtesy of the Nuclear Waste Management Organization (NWMO) of Canada.
Figure 3.1: Geological map of Southern and Eastern Ontario (OGS 1991) showing approximate locations of the samples considered in this study and the three main projects; the Niagara Tunnel Project (NTP), the Deep Geological Repository (DGR) and the Ottawa Light Rail Tunnel (OLRT).
These projects are discussed in more detail in Chapters 5, 7 and 9. The sample locations are shown in Figure 3.1, in relation to the surficial bedrock geology.

The majority of the samples are carbonates and mudrocks (shales and siltstones), which are the most common rock types found within the subsurface, as indicated in Figure 3.1. The carbonates are strong (UCS > 75 MPa), resistant to erosion (excluding dissolution which forms karst) and form topographic highs; for example the edge of the Niagara Escarpment is formed by the Lockport Formation in the Niagara Region. Mudrocks are easily eroded and typically form low land areas. For example, Queenston shale swells and deteriorates rapidly when exposed to fresh water (Rigbey and Hughes 2007).

Construction projects in Ontario are typically close to surface (< 200 m deep). However, some deep exploration boreholes have been drilled for oil and gas in Southern Ontario, and two boreholes (labelled DGR-2 and DGR-4) reaching down to the Cambrian have recently been drilled at the Bruce Nuclear Power Plant for the proposed Deep Geological Repository for Low and Intermediate Level Waste. The data included in this research is representative of the major rock units of Ontario.

In this research, correlations are made between field and intact rock test results, in an attempt to understand the variation in properties found on various scales; within rock types, formations, and construction sites. Understanding this variability will be useful for the preliminary design stage of future engineering projects.

3.2 Field and Laboratory Testing Techniques
Sedimentary rocks in Ontario were deposited in the Michigan, Appalachian and Ottawa Embayment basins, and the bedding is generally sub-horizontal. Deep geotechnical boreholes are typically drilled vertically, and the majority of test data are therefore obtained from horizontally-
bedded samples. However, in certain cases inclined boreholes have been drilled to investigate the presence of vertical structure.

Engineering investigation programs typically consist of field and laboratory tests which determine the intact and rock mass properties. For underground projects this is typically performed using core samples from surface boreholes. Sampling should be conducted to collect representative samples from all formations that will be encountered during excavation, and be of an appropriate size for testing. The ISRM suggested method (Fairhurst and Hudson 1999) for uniaxial compressive strength recommends that the core diameter should be no less than 50 mm. Various field and laboratory tests commonly used for sedimentary rocks are listed in Table 3.1. Some tests, such as for swell, are specific to sedimentary rocks, while others are common for all rock types.

Field tests should be conducted as soon as possible after the core has been recovered. Laboratory samples should be handled carefully, and be efficiently prepared and sealed for transport to the laboratory testing facility, particularly for samples which may be sensitive to changes in moisture content.

3.2.1 Factors Affecting Sample Quality
The sample quality can be affected during drilling, when collecting the sample, during transport, and during preparation for testing. There are guidelines for preserving samples; the ASTM standard D5079 (ASTM 2008a) describes procedures used for preserving samples for transportation and testing.

Sample quality can be influenced during drilling and extraction. There are many drilling options depending on the rock type being investigated. Generally, the quality is more affected while it spins inside of the core barrel. Inner tubes, inside the core barrel are used for collection.
Table 3.1: Common field and laboratory tests conducted during field investigation programs for underground projects.

<table>
<thead>
<tr>
<th>Test</th>
<th>Description</th>
<th>Reference / Standard</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Field</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Point Load</td>
<td>Preliminary index used to aid in classifying and characterizing the rock with respect to strength</td>
<td>Franklin (1985) D5731-08 (ASTM 2008a)</td>
</tr>
<tr>
<td>Slake Durability</td>
<td>Index value of the susceptibility of rock materials to degrade during wetting and drying cycles</td>
<td>Franklin and Chandra (1972) D4644-08 (ASTM 2008b)</td>
</tr>
<tr>
<td>Ultrasonic Pulse Velocity</td>
<td>P- &amp; S-wave velocities are used to determine the dynamic elastic constants of the rock</td>
<td>Rummel and Heerden (1978) D2845-08 (ASTM 2008c)</td>
</tr>
<tr>
<td>Brazilian Tensile Strength (BTS)</td>
<td>Can be determined in the field, more commonly in the laboratory</td>
<td>Bieniawski and Hawkes (1978) ASTM D3967-08 (ASTM 2008d)</td>
</tr>
<tr>
<td>Swelling Test</td>
<td>Variety of methods available to determine susceptibility to expand</td>
<td>Madsen (1999) D4644-08 (ASTM 2008b)</td>
</tr>
<tr>
<td><strong>Laboratory</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Unconfined Compressive Strength (UCS)</td>
<td>Standard strength test for geomechanics, used as input into numerical models</td>
<td>Fairhurst and Hudson (1999) ASTM D7012-10 (ASTM 2010)</td>
</tr>
<tr>
<td>Crack Initiation (CI)</td>
<td>Determined from strains or acoustic emissions during UCS testing</td>
<td>Diederichs and Martin (2010)</td>
</tr>
<tr>
<td>Crack Propagation (CD)</td>
<td>Determined from strains or acoustic emissions during UCS testing</td>
<td>Diederichs and Martin (2010)</td>
</tr>
<tr>
<td>Direct Tensile Strength (DTS)</td>
<td>Direct tensile resistance to sample being pulled apart</td>
<td>Bieniawski and Hawkes (1978) ASTM D2936-08 (ASTM 2008e)</td>
</tr>
<tr>
<td>Brazilian Tensile Strength (BTS)</td>
<td>Indirect tensile resistance</td>
<td>Bieniawski and Hawkes (1978) ASTM D3967-08 (ASTM 2008d)</td>
</tr>
<tr>
<td>Triaxial Compression</td>
<td>Confined, biaxially or triaxially, strength test</td>
<td>ISRM (1983) ASTM D7012-10 (ASTM 2010)</td>
</tr>
<tr>
<td>Swelling potential</td>
<td>Value for swelling susceptibility</td>
<td>Lee and Lo (1989)</td>
</tr>
</tbody>
</table>

There are three types of inner tubes used in the core barrels to extract core; single, double, and triple, and these constitute the part of the core barrel that retrieves the core. The ASTM D2113-08 (ASTM 2008b) standard describes the different tubes and their use in collecting rock samples for testing. A single tube is the simplest design and is suitable for use in massive hard
rock. However, its disadvantage is that the tube and core can rotate within the tube during drilling, and thus the core sample can be affected. This can therefore cause deterioration of samples during the drilling process, and the length of intact core recovered can be reduced by breakage. A double tube seals the core within the tube to minimize water circulation around the core during drilling. However, circulation at the bottom of the tube can cause deterioration, but this risk is greatly reduced compared to use with a single tube. Double tubes can be solid or split. A split barrel enables easier extraction of the core from the tube and minimizes damage to the core. With a solid tube damage to the core can occur if it is removed forcefully in the event of it becoming jammed within the tube. The advantage of the double tube is that the inner tube remains stationary during drilling; thus minimizing potential damage when the core spins within the tube. According to the ASTM standard (ASTM 2008b), this type of tube is typically the minimum requirement for drilling investigations where varying conditions may be encountered. The triple barrel is similar to a double tube, but has an inner liner which is removable from the outer barrel and tube. It can be split or solid, and increases efficiency in handling and logging the core. Choosing appropriate drilling equipment that is able to deal with the rock that may be encountered can increase the efficiency of the drilling program and minimize damage to the core. Depending on the rock type, damage can occur due to changes in insitu confinement and/or the moisture content.

Lim and Martin (2010) suggest that core disking is initiated when the maximum principal stress normalized by the tensile strength is around 6.5 (for granitic rocks). Disking is evident when closely-spaced fractures occur within the drill core. However, oriented micro-damage (a precursor to disking) may not be visible. This micro-damage can influence testing results and the orientation may affect different tests in different ways. Rocks with swelling potential deteriorate
rapidly under changing moisture conditions (Rigbey and Hughes 2007) and may result in physiochemical disking. Although core disking can not necessarily be avoided, proper sample sealing once the core is extracted can minimize changes in moisture content and therefore damage to the samples prior to testing.

A comparison between geophysical P- and S-wave velocities of the borehole and those of the laboratory samples can be performed to determine if damage has been caused to samples prior to testing. A comparison for rocks from Ontario is shown in Figure 3.2.

Borehole velocities were taken as the average of three point measurements over a distance of 0.06 m. Single points and the average of five points over 0.18 m were both examined, with the three point average giving the most consistent result. The borehole and laboratory measurements are compared in Figure 3.2 and are separated by main rock types. The dolomite samples plot closest to the one to one line indicating that these samples have similar borehole and laboratory velocities, both P- and S-wave. There is more scatter for the limestone samples, which in this case are typically argillaceous in nature. It is likely that the limestone data points furthest from the one to one line have the highest argillaceous content make these sample more susceptible to micro cracking during transport. The shale points show the most scatter, however, there are many samples which plot close to the one to one line indicating that good sample transport and preparation techniques have been used. It should be noted that this comparison is not an exact measurement of damage to laboratory samples, but rather a potential indicator when there is a large deviation from a one to one relationship. This is examined in more detail in section 3.3 which also examines how the intact properties are affected.
Figure 3.2: Comparison between P- and S-wave velocities from boreholes (BH) and from the laboratory samples (Lab) prior to testing for the main rock types (field results courtesy of the Nuclear Waste Management Organization and laboratory results from Gorksi et al. 2009, 2010, 2011).

3.2.2 Field Testing

To gain an understanding of the rock mass, it is common practice to conduct index tests in the field during investigation programs. As listed in Table 3.1, the common field index tests used include swell testing, slake durability, and point load.

Swell testing is discussed here because only free swell test results are presented and the equipment for such tests can easily be erected in the field, following the ISRM suggested method.
(Einstein 1989). Rigbey and Hughes (2007) describe various swelling cell arrangements which can be employed. Lee and Lo (1989) define the swelling potential as being the slope of time versus the swelling strain graph between 10 and 100 days. During a swelling test, the sample breaks down slowly as the sample expands, due to the swelling process.

The swelling process is also a mechanism which breaks down susceptible samples during slake durability testing, which should be conducted according to the ASTM standard test method D4644-08 (ASTM 2008c). This test measures the resistance of the sample to wetting and drying cycles, and is intended for use as an index to compare rocks (Franklin and Chandra 1971). The index value is a measure of the percentage by dry mass of the sample retained on a 2 mm sieve following wetting and drying cycles. Results from the DGR site were reported by Gaines and Sterling (2009ab), and as expected the more argillaceous formations have a lower index value, which indicates such formations have the potential to disintegrate over time if exposed to changes in moisture. With limited data available, the slake durability test results were not correlated to other index values or properties.

The point load index is a common field test which can be correlated to compressive and tensile strengths (these correlations will be discussed in more detail in section 3.3). The point load test is intended as an index to help with the strength classification of rock mass (Franklin 1985). The point load is determined as the load at failure, $P$, divided by the squared equivalent core diameter, $D_e^2$. The equivalent core diameter is used when irregular or roughly square samples are tested, and otherwise the squared core diameter is used as the divisor. The point load test can be conducted at the drilling site as a means of understanding the various zones of rock strength.
Field testing can be conducted in a quick and efficient manner during the drilling campaign, and field index testing can be used in preliminary design studies to determine which areas require a more detailed investigation and laboratory testing.

### 3.2.3 Laboratory Testing

The most common laboratory test conducted is the Uniaxial Compression test, which yields the ultimate strength or Unconfined Compressive Strength (UCS) and modulus of intact rock. The modulus is typically determined at 50% of the intact strength and called the Young’s Modulus, $E_i$. The estimation of crack damage thresholds (other than peak strength) using strain or acoustic emission-based methods is becoming more common in routine UCS testing, as it can be performed with minimal additional effort. Diederichs and Martin (2010) summarize the methods available for measurement of crack damage thresholds, and these include crack initiation (CI), and crack propagation (CD). The different methods used to determine damage thresholds are illustrated in Figure 3, and are only briefly discussed here for clarity.

CI represents the first onset of newly distributed grain-scale cracks within the sample during testing, and the CI threshold can be determined as the point where the stress-strain (volumetric or lateral) deviates from linearity (Brace et al. 1966; Bieniawski 1967; Lajtai and Lajtai 1974).

Tapponier and Brace (1976) found that natural micro-cracks occurring below the CI threshold are generally limited to the grain-scale for crystalline rocks, and if pre-existing damage (micro-cracks) exists in the sample then linear-elastic behaviour may not be present, or the exact onset of crack initiation may not be accurately defined. Crouch (1970) suggested that the lateral strain-axial strain plot could be used to determine CI. However, both the lateral and axial strains around the CI can change in a similar fashion during compression tests, and therefore mask the
true CI threshold due to a delay in the occurrence of the deviation from linearity. The CI threshold can also be determined as the deviation from linearity of the Inverse Tangent Lateral Stiffness (ITLS) versus the applied stress plot (after Ghazvinian, 2010), and because the rate of change in the slope of the axial stress-lateral strain curve is examined the ITLS method avoids the problems stated above.

The CD threshold is the point where cracks begin to occur in an unstable manner (Bieniawski 1967) and when the cracks begin to interact. In an unconfined compression test, CD can be determined as the deviation from the linear elastic response of the strain in the direction of loading.

![Figure 3.3: Methods of determining Crack Propagation (CD) and Crack Initiation (CI) thresholds from unconfined compressive strength (UCS) tests. Peak = peak strength, AE = acoustic emission, and Vol. = volumetric strain.](image)

<table>
<thead>
<tr>
<th>CD</th>
<th>Vol. Strain reversal</th>
<th>Next rapid increase in AE Hit Count</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial Strain non-linearity</td>
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</tr>
<tr>
<td>CI</td>
<td>Lateral Strain non-linearity</td>
<td>Crack Vol. Strain reversal</td>
</tr>
</tbody>
</table>

80
CI and CD thresholds can also be detected using acoustic emission (AE) sensors, and changes in the rate of acoustic emissions during testing have been found to correlate well with the damage thresholds discussed above (e.g. Scholtz 1968; Lockner 1993; Falls 1993; Eberhardt et al. 1998), as illustrated in Figure 3.3. These values are important parameters for input into numerical models.

Engineering studies often focus on the unconfined compressive strength (UCS) of intact rock samples, and the direct tensile strength (DTS) is often overlooked because of difficulties in preparing samples; many poorly-prepared samples fail invalidly with the failure surface not passing through the middle of the sample and thus must be discarded. Shale samples are the most difficult to prepare for testing using the recommended dog-bone DTS shape (Brace 1964, Hoek 1964). Samples for indirect tensile methods such as the Brazilian Tensile Test (which can be conducted in the field but are more commonly completed in the laboratory) are much easier to prepare, although invalid tests (when fracture is not through the middle of the sample) are also frequent and each sample should be examined after testing to determine its validity (Colback 1966).

Results of properties from laboratory analyses and from other sources of rock mass information can be used to classify the rock mass into zones of expected behaviour, and in addition such properties can be used in numerical models to evaluate various design aspects. It is therefore important to determine the properties necessary for use in further analyses in relation to design requirements.

3.2.4 Analysis of Data
Field and laboratory data collected from literature and presented as part of this study were analysed. Analysis commenced using individual test results, and these were examined based on
appropriate geological classification systems. Dunham’s (1962) system was used for carbonates and Lundegard’s and Samuels’ (1980) for mudrocks. Sandstones were not further classified into different sub-groups as there was relatively few samples in comparison to carbonates and mudrocks.

In the literature, samples were not necessarily classified by the original authors, and in such cases, where details were available, the results were classified using borehole reports (or other geological reports) using the same location, ensuring that sample depths were matched with geological descriptions of similar depth. It was considered that misclassification of some of the samples was possible using this method, as even at the scale of a UCS sample (typical 150 x 54 mm) variable grain sizes or classes are still possible. The misclassified samples result in outliers, which have been assessed on an individual basis, and have not been removed to highlight the importance of classifying sedimentary rocks.

A correlation analysis between field and laboratory results was conducted, together with the correlation between different laboratory results. These findings are discussed after first discussing individual field and laboratory test results.

3.2.5 Field Testing Data
Swelling rocks in Ontario (such as the Queenston or Georgian Bay shales) are commonly located near the surface. Micic and Lo (2009, 2010) reported the swell potential for a number of different rock formations and these have been plotted against the calcite content in Figure 3.4. The results indicate that swell potential is reduced with an increase in calcite content, and that vertical swell potential is generally greater than horizontal potential. The Salina and Cabot Head formations exhibit the highest swell potential, although certain horizons of the Salina Formation contain a high percentage of calcite. It is possible that other formations swell, however this possibility has
either been considered negligible, as with the Cobourg limestone (Micic and Lo 2010), or has not yet been determined by study.

Point Load Index values were sorted into main rock types; dolomite, limestone, argillaceous limestone, shale and sandstone. However, due to limited data a statistical analysis could not be conducted on the formation scale, or on classified data. The results have therefore been examined to determine the range and variability of the index values for the different rock types. The results were separated into carbonates (Figure 3.5 top), and siliciclastics (Figure 3.5 bottom), and the ranges were determined as follows: Dolomite: 0.15 to 8.85 MPa; Limestone: 1.19 to 10.27 MPa; Argillaceous Limestone: 2.61 to 10.03 MPa; Shale: 0.02 to 11.43 MPa; and Sandstone: 0.20 to 2.70 MPa.

![Figure 3.4: Decrease of swell potential with increase of calcite content of formations from the DGR site (data from Micic and Lo 2009, 2010, and Perras 2009).](image)
Figure 3.5: Point load index results for main rock types in Ontario (data from Hudec and Russell 1981, Dusseault and Loftsson 1985, Russell 1993, Gaines and Sterling 2009ab, and Golder and Associates 2011).
With the exception of sandstone, the range (minimum to maximum) is shown to be similar for all rock types. However, sandstone tests were conducted by Russell (1993) on the Sylvania Formation samples from the Windsor area, and it is reported that these have special engineering properties. A study by Stump (1980) shows that this sandstone explodes into loose sand during uniaxial compression tests, and Russell (1993) suggests that this is related to the grains becoming concave during compaction and diagenesis. The grains rotate during failure with little shearing because of their interlocked nature and the high strength of the quartz grains.

The cement binding the sandstone grains has a variable dolomitic content, making it weaker than the grains and therefore controlling the failure process. The Sylvania sandstone is much weaker (average <50 MPa in UCS) than other sandstones in Ontario, such as the Thorold or Whirlpool sandstones (average UCS >140 MPa).

The carbonate results show smaller coefficients of variation (COV) than those of the siliciclastics. This is possibly related to the more consistent mineralogy of carbonates. High strength or low strength interbeds in the siliciclastic samples could lead to outliers which are not consistent. The level of geological detail was insufficient to determine the interbed influence. The entire argillaceous limestone results are derived from the Cobourg or Lindsay formations (Gaines and Sterling 2009ab, Golder and Associates 2011). Despite variable interbeds of other argillaceous rock, the overall calcite content is still high (>74% from Micic and Lo 2010). Correlations between field and laboratory measurements will be discussed after the results of individual laboratory testing below.

3.2.6 Laboratory Testing Data
Similar to the field testing data, the laboratory data were divided into siliciclastic and carbonate rocks. The number of UCS data points allowed for a statistical evaluation using the combined
siliciclastic-carbonate classification system (Figure 2.11). Limited tensile strength data were available in the literature, and only main rock types could be evaluated. In order to evaluate the range of each class, the data were first plotted in histograms; with UCS data shown in Figure 3.6, CI data shown Figure 3.7, modulus data shown Figure 3.8, and tensile data shown Figure 3.9.

Due to the large number of test results, both from the literature and this study, the references have not been listed with each figure, but rather have been listed in Table 3.2 for data from the literature, in Table 3.3 for data from the DGR site, and in Table 3.4 for data tested as part of this study, along with the corresponding number of samples for each test type.

Generally, the finer-grained siliciclastic test results show narrower ranges than the coarser (sandstone) siliciclastics and carbonate rocks. The test results are typically normally-distributed, with the exception of CI and tensile strength for shale, and the tensile strength of the sandstone. There is significantly more data available for mudrocks and carbonates than for sandstone and siltstone, which is related to surface proximity, as previously mentioned. The test results are discussed in further detail by test-type, and significant differences between the classes are discussed in the following paragraphs.

A wide range of UCS results are shown (Figure 3.6) for the sandstone and carbonate rocks. As previously mentioned, Sylvania sandstone is generally weaker than other sandstones in Ontario. The mean Sylvania (dolomitic cement (Russell 1993)) UCS was found to be 44 MPa. The Whirlpool and Thorold sandstones (non-calcareous cement (Armstrong and Carter 2010)) have mean UCS values of 144 MPa and 146 MPa, respectively. The high strength sandstones account for the two main peaks (bimodal) in the sandstone distribution of UCS results.
Figure 3.6: Comparison of UCS histograms for siliciclastic (top) and carbonate (bottom) rocks.
Figure 3.7: Comparison of CI histograms for siliciclastic (top) and carbonate (bottom) rocks.
Figure 3.8: Comparison of $E_i$ histograms for siliciclastic (top) and carbonate (bottom) rocks.
Figure 3.9: Comparison of tensile strength (direct or indirect as indicated) histograms for siliciclastic (top) and carbonate (bottom) rocks.
Table 3.2: Number of samples for each test result listed by source and rock class.

<table>
<thead>
<tr>
<th>Source</th>
<th>Rock Class</th>
<th>UCS</th>
<th>CI</th>
<th>CD</th>
<th>BTS</th>
<th>Ei</th>
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</tr>
<tr>
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<td>Shale</td>
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Table 3.3: Number of samples for each test result extracted from individual DGR testing data reports (Gorksi et al. 2009, 2010, 2011).

<table>
<thead>
<tr>
<th>Source</th>
<th>Rock Class</th>
<th>UCS</th>
<th>CI</th>
<th>CD</th>
<th>BTS</th>
<th>E_i</th>
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Table 3.4: Number of samples for each test result listed by rock class from data presented as part of this study.

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<th>Rock Class</th>
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<th>CI</th>
<th>CD</th>
<th>DTS¹/BTS²</th>
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<td>27²</td>
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The UCS distribution for the siliciclastic mudstones shows a bimodal distribution (Figure 3.6). According to Brogley et al. (1998) the Queenston formation ranges from siltstone along the eastern side of Lake Ontario, to shale (claystone) along the shore of Lake Huron. The data from the Niagara Region, where specific grain size information was not available, were classified as mudstones based on the author’s experience during the Niagara Tunnel Project (Perras 2009).

Limited data from a borehole for the Niagara Tunnel shows corresponding grain size information for the UCS tests in the Queenston, which indicates that claystones to siltstones are present, based on the silt fraction (Figure 3.10). The increasing silt fraction causes an increase in the UCS, although with a wide scatter. Interestingly, the CI value is largely unaffected by the silt fraction until greater than 70%. This aspect will be discussed later.

The clear bimodal distributions in the siliciclastic UCS data are not present in the carbonates, which show much wider distributions that appear to have similar mean values and ranges (Figure 3.6). For the siliciclastic rocks, the sandstones show the most obvious bimodal distribution. The lower peak, for both UCS and $E_i$, is close to the finer siliciclastic peaks and could show the influence of heterogeneity on the test results. These samples could also be misclassified in the original literature and would perhaps be better described as siltstones. Both the crystalline limestone and the lime packstone show a weak bimodal distribution. This could partly be related to the fact that each carbonate class plotted in Figure 3.6 is dominated by one formation. Similar weak bimodal distributions were present in the point load results (Figure 3.5). However, this does not explain the wide range of values.

For the crystalline dolomite, the uniform distribution may reflect the varying alteration of the original limestone material by the dolomitization process. Morrow (1990) describes a variety of dolomitization models that all include three main components. The first is the availability of
magnesium to replace calcium in the limestone. Secondly, a transportation mechanism must be present to transfer magnesium and calcium. Finally, the solution (water) must be conducive to the chemical reaction which transforms calcium carbonate to magnesium carbonate.

The dolomitization process does not necessarily replace calcium carbonate in a uniform manner, and therefore the sample strength may be more similar to the type of original limestone, or close to a fully-dolomitized rock (or anywhere in between), depending on the degree of replacement.

Figure 3.10: Changes in strength thresholds (CI, CD, and UCS) with increasing amounts of silt-sized grains in the Queenston Formation, where 0% siltstone is a pure clayshale.
Similar distributions of the UCS values of the carbonate classes could partly be due to the greater uniform mineralogy (i.e. calcium carbonate). Both the grains and the matrix are dominated by calcium carbonate, and as such the mechanism of failure is not associated with differences in mineralogical makeup of the samples (i.e. different grain stiffnesses). The heterogeneity in carbonate rocks lies in the grain size and the percentage of grains versus cement. These effects may still be present, as indicated by the higher mean strength of the lime mudstone over the other classes, despite uniform mineralogical make up.

The CI distributions for the siliciclastics are similar, and this is reflected by the fine-grained nature of the samples available in the literature (Figure 3.7). However, sandstone CI values were not abundant in the literature at statistically significant volumes. The six sandstone values available have an average of 81 MPa, which is almost four times larger than the other siliciclastics. These sandstones include the Whirlpool, Power Glen, and Grimsby formations, which are all strongly cemented with silica. The weaker matrix materials of the mudstones and shales control the CI threshold. In Figure 3.10, the CI threshold for the Queenston Formation is consistent with the increase siltstone within the sample (up to roughly 75 %). At a value of 80 % siltstone, the CI value doubles, demonstrating the weak-link concept for CI.

During compression or tensile tests the CI value is an indicator of the point when cracks begin to occur in the sample. In a layer cake of shale and siltstone, the cracks begin in the shale layer, but are suppressed in the siltstone layer until sufficient stress is accumulated in the sample to cause the cracks to initiate and propagate in and through the siltstone layers. This peak strength also increases with increasing amounts of siltstone, as there is a greater volume for crack absorption in the sample during loading.
For carbonates, the matrix and grains are similar in strength and the CI value is therefore controlled by other factors, such as flaws at the grain boundaries, or the grain size and shape. Initial flaws in carbonate rocks include mud-carbonate interfaces, fossil fragments, or pores. Different grain stiffnesses can also induce tensile stress at the grain to grain contact, inducing a fracture. Different grain stiffnesses can arise in carbonate rocks through the processes of recrystallization or dolomitization, where grains are gradually changed into either a different crystal structure, or a different composition, respectively. Recrystallization and dolomitization can also create void space, which is a potential site for fracture initiation.

The stiffness distributions (Figure 3.8) for the siliciclastic rocks are similar to the UCS and CI distributions (Figures 3.6 and 3.7), with the siltstones having a narrow range and the sandstones having the widest range. As with the UCS values, the sandstones have a bimodal distribution due to the weaker Sylvania formation measurements. The moduli of the carbonates still have a wider range than the siliciclastic rocks, however in contrast to the UCS and CI distributions the crystalline dolomite has a wider modulus range than the other limestone classes. Again, this maybe the result of the varying degrees of dolomitization.

Tensile strength measurements used in the literature were dominantly indirect Brazilian tests. Direct tensile data (courtesy of Ontario Power Generation) on shale formations are log-normally distributed, as shown in Figure 3.9. In Ontario the bedding is generally horizontal, which influences the failure mechanism during tensile testing. The failure plane in a DTS test from vertical boreholes will be parallel to the bedding, and thus result in weaker tensile values. In contrast, BTS failure planes in cores from vertical boreholes will be perpendicular to bedding. For the carbonate rocks, all test data was from BTS tests. The results show different mean values of normal fits to the data (Figure 3.9). The crystalline dolomite and limestone values are greater than
the mudstone values. The crystallization process fuses the grains together, making the rock stronger, however it can also leave voids which become crack initiation sites and this contributes to the wide range of values for the crystalline samples (Figure 3.9).

Over all, laboratory properties show different trends between the siliciclastic and carbonate rock types. Grain size and type play an important role in the failure process, as well as the matrix material. Typical correlations between field and laboratory results, and between different laboratory results, will be discussed in the next section to gain a better understanding of the influences on the laboratory test values.

3.3 Correlations between Properties
While awaiting laboratory results, measurements from the field can be used for preliminary design purposes. Relationships comparing field measurements with laboratory measurements can then be helpful in predicting values. In the literature, P- and S-wave velocities, and point load index values were the most commonly reported field measurements.

The P-wave velocity relationship to the UCS was examined and there is some correlation (Figure 3.11). The P-wave velocities of the various classified rock types are similar, and independent trends had less correlation than the all the data combined. With respect to the relationship between down borehole and laboratory P-wave values (Figure 3.2) which showed that a one to one relationship exists for more competent rock types, the P-wave velocity could be used as a preliminary method for estimating the UCS value, however there is no distinct separation of the rock classes (Figure 3.11). This means that the predicted UCS value is non-unique and cannot be used as a predictive tool to determine rock type only the potential strength. This may still be useful for feasibility studies. Due to the steep slope at high velocities (> 5.5...
km/s in Figure 3.11) the preliminary estimate becomes less reliable due to the wide range of potential UCS values (> 150 MPa) for a narrow velocity measurement.

A better method for estimating the UCS is to use the point load index. The average point load index was therefore plotted against the average UCS for different formations. A strong linear relationship can be seen between the average values (see Figure 3.12), and this is in agreement with many of the relationships found in the literature, as summarized by Zang (2005). Average point load values should also be used to minimize the error in the prediction, as there is wide scatter in the maximum values shown in Figure 3.12. Other correlations between field and laboratory values are reported in the literature (examples summarized by Zang 2005). However, insufficient values were available to make further correlations.

![Graph showing the relationship between P-wave velocity, Vp, and UCS for different rock classes.](image)

**Figure 3.11:** Relationship between P-wave velocity, Vp, and UCS for different rock classes.
A correlation between laboratory measurements is an effective means to verify that the rock properties are within the expected range of values. If the laboratory results lie outside the expected range, then further investigation can be made to verify the results.

Stiffness is used for a common correlation with uniaxial compressive strength, and is known as the modulus ratio (MR). Hoek and Diederichs (2006) report modulus ratios for siliciclastic rocks which are similar to those found in this study (Table 3.5). There is less agreement between the carbonates, with those from this study being lower than the range reported by Hoek and Diederichs (2006) (as shown in Table 3.5). Also reported in Table 3.5 are estimates from this study for Lime wackestones and Lime grainstones, which are based on limited data.
Table 3.5: Modulus ratios for sedimentary rocks reported by Hoek and Diederichs (2006) after Deere (1968) and Palmstrom and Singh (2001) compared to modulus ratios found for the sedimentary rocks of Ontario. The present study results are for one standard deviation about the mean. * indicates terminology used in this study and ** indicates estimated range due to lack of statistically representative data for Ontario.

<table>
<thead>
<tr>
<th>Class</th>
<th>Texture</th>
<th>Hoek and Diederichs (2006)</th>
<th>Ontario (present study)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silticlastic</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sandstone</td>
<td>200 – 350</td>
<td>200 – 350</td>
<td></td>
</tr>
<tr>
<td>Siltstone</td>
<td>350 – 400</td>
<td>200 – 400</td>
<td></td>
</tr>
<tr>
<td>Mudstone*</td>
<td>-</td>
<td>200 – 400</td>
<td></td>
</tr>
<tr>
<td>Claystone</td>
<td>200 – 300</td>
<td>150 – 450</td>
<td></td>
</tr>
<tr>
<td>Carbonate</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Micritic limestones (lime mudstones*)</td>
<td>800 – 1000</td>
<td>200 – 650</td>
<td></td>
</tr>
<tr>
<td>Sparitic limestones</td>
<td>600 – 800</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Lime wackestone**</td>
<td>-</td>
<td>300 – 500</td>
<td></td>
</tr>
<tr>
<td>Lime packstone*</td>
<td>-</td>
<td>200 – 500</td>
<td></td>
</tr>
<tr>
<td>Lime grainstone**</td>
<td>-</td>
<td>100 – 300</td>
<td></td>
</tr>
<tr>
<td>Crystalline limestones</td>
<td>400 – 600</td>
<td>250 – 500</td>
<td></td>
</tr>
<tr>
<td>Dolomites (crystalline dolomite*)</td>
<td>350 – 500</td>
<td>250 – 600</td>
<td></td>
</tr>
</tbody>
</table>

The distribution of the MR’s for the classified data can all be fit within a normal distribution, with the exception of crystalline dolomite, which is lognormal. Otherwise the lognormal fits are similar to the normal fits for the remainder of the classes. Histograms of the data, and the best fit distributions, are shown in Figure 3.13. It should be noted that ±1 standard deviation was used in the reported range of MR’s in Table 3.5.

The MR can be used in conjunction with UCS values estimated from point load during preliminary investigations and design, to determine intact moduli for the rock in question. The rock mass modulus can then be estimated using the approach suggested by Hoek and Diederichs (2006).
Figure 3.13: Modulus Ratio (MR) distributions for (top) siliciclastic and (bottom) carbonate rocks in Ontario.
The conceptual model of Diederichs (2007) is used to describe the brittle behaviour of the rock mass near an excavation boundary, and requires the CI threshold to determine the peak failure envelope. Many researchers have found that the CI value typically lies within 30%–50% of the UCS of the intact rock (Brace et al. 1966, Lajtai and Dzik 1996, Pestman and Van Munster 1996, Diederichs 2003).

The dolomite and limestone data for Ontario fall within the common range of CI / UCS ratios (Figure 3.14), with the mean of the dolomite being slightly less than the limestone based on a normal fit to the data (38% < 39%). The wide range of the siliciclastic normal fit is due to the high number of values at 53%.

Figure 3.14: CI to UCS ratios for the main rock types in Ontario.
These values come from the Queenston Formation, and may represent samples with interbedded limestone or sandstone, since both the CI and UCS values are much higher than the typical values for the Queenston. The carbonate content of the Queenston, including disseminated crystals in the shale matrix and interbeds of limestone, increases to the northwest away from the Taconic source zone, and it is reported that the lower part of the Queenston consists of thinly interbedded and interlaminated siltstone, sandstone, and limestone with red and green shale (Armstrong and Carter 2010).

Generally the results and correlations presented above are in agreement with similar rocks from other locations outside Ontario. It is noted that the correlations are only useful for predicting preliminary values when field or laboratory testing has not yet been completed. It is also necessary to understand the variability of properties from one location to another when utilizing the above correlations for predicting preliminary values.

3.4 Discussion on Predictability
The most commonly tested rocks in Ontario are those of the Middle and Upper Ordovician, due to their proximity to the ground’s surface in major cities (e.g. Toronto, Ottawa). The average and range of laboratory properties for each formation are summarized in Figure 3.15 for UCS and CI, and in Figure 3.16 for E, and T. The number of samples tested is indicated in Tables 3.2, 3.3, and 3.4, which should be considered when assessing how representative the range of values is for each formation. In some cases a wide range of values have been stated in the literature, which reflects the need to accurately classify each sample being test not just the larger interval over which the samples may come from in the borehole. On the low end of the range might be the weak end member, shale for example, and on the high end of the range might be the strong end member, sandstone for example, of an interbedded formation.
Figure 3.15: UCS and $E_i$ averages and ranges for the various formations across Ontario. Where possible, formations have been classified into different rock types, as indicated and this applies to both UCS and $E_i$ measurements. For main rock types refer to Figure 3.1.
Figure 3.16: CI and BTS averages and ranges for the various formations across Ontario. Where possible, formations have been classified into different rock types, as indicated and this applies to both UCS and $E_i$ measurements. For main rock types refer to Figure 3.1.
This heterogeneity can be captured in a good description of each sample to be tested, but was found to not be reported explicitly in most of the published sources. Two of the most widely tested formations, the Queenston and Lindsay/Cobourg, are examined in more detail to determine how well the properties compare from different locations.

3.4.1 The Queenston Formation

Although numerous UCS tests were conducted on the Queenston for both the Niagara Tunnel Project (NTP) and the DGR, as shown in Figure 3.17, only a limited number of the completed tests included volumetric strain measurements (from Niagara), which were used to determine CD and CI. The volumetric strain reversal and the onset of the non-linearity of lateral strain points were used to determine the CD and CI thresholds, respectively. The values for the DGR were taken directly from various testing reports (Gorski et al. 2009, 2010, 2011). For clarity, only UCS and CI are plotted in Figure 3.17, with respect to the depth datum and percentage of siltstone content.

The strength values in Figure 3.17a are plotted using the top of the Queenston Formation as the datum. At first inspection it seems that there is a wide range of strength values for both the NTP and DGR. However, Figure 3.17 indicates a wider range at the DGR site. Values of, UCS, and CI increase with depth (Figure 3.17b), and it is likely that this partly relates to the transition to the Georgian Bay Formation at the base of the Queenston, which contains a greater number of siltstone and limestone interbeds. This would also account for the increasing stiffness with depth at the DGR site (Figure 3.18). The Queenston Formation is over 300 m thick at the NTP site. There are a few UCS values from the NTP below 100 m comparable to the upper range of the DGR values. The CI and \( E_i \) maximum values from the NTP are lower than those at the DGR and could be related to the current depth below the ground, which is much greater at the DGR site.
Figure 3.17: Comparison of a) UCS and b) CI between the NTP and DGR for Queenston Formation test results.
Figure 3.18: Comparison of $E_i$ between the NTP and DGR for Queenston Formation.

The UCS values in the upper portion of the Queenston Formation at the NTP site show a wide range, which is generally consistent with increasing depth. However, a closer examination indicates that there are potentially three strength bands, which all exhibit increasing strength with depth. The first band is at 0–25 m, whereas the second and third have depth ranges of 25–75 m and 75–100 m, respectively. Similar bands can be seen in the CI thresholds, and these are likely to be related to changes in the depositional environment.

The carbonate content of the Queenston, including the disseminated crystals in the shale matrix and interbeds of limestone, increases to the northwest away from the Taconic source zone. It has been reported that the lower part of the Queenston consists of thinly interbedded and
interlaminated siltstone, sandstone, and limestone, with red and green shale (Armstrong and Carter 2010). Thus, the increasing strength and stiffness with depth and distance from the source could be associated with an increase in the calcite content. The wide range may also reflect variations in the calcite content at similar depths since the calcite content may not be uniform at a single horizon.

For the project, a limited number of samples from the NTP were used to determine the percentage of siltstone and shale. Examining the UCS values with respect to the siltstone percentage shows that the UCS generally increases with increasing siltstone content, whereas the CI value is largely unaffected below 80% (Figure 3.10).

Amann et al. (2011) investigated crack initiation in the Opalinus clayshale, from Switzerland, and indicated that tensile cracks began in the stiffer layers, and shear cracks began in the softer layers as a result of the stiffness contrast. In the case of the Queenston, the siltstone layers are stiffer and, according to the theory postulated by Amman et al. (2011) would be the area where cracks first occur. Because CI has been determined as the point where the lateral strain deviates from linearity, the data suggest that the lateral strain deviation is controlled by the presence of the siltstone, irrespective of the percentage (up to 80%). The lateral stiffness is controlled by the shale layers (even at higher siltstone content), and because the stiffness of the siltstone is incompatible with the shale, tensile cracks develop in the siltstone. Above 80% siltstone, a sample’s lateral stiffness is controlled by siltstone (which is stronger), and this results in a high CI. The siltstone layers absorb cracks during loading that cannot propagate through the shale layers, and this influences the peak strength. With increasing siltstone content, this peak strength also increases because there is a greater volume for crack absorption in the sample.
during loading. The layering also gives rise to anisotropic strength and stiffness, but does not influence CI, as mentioned previously.

3.4.2 The Lindsay / Cobourg Formation
At a depth of 680 m, the Cobourg Formation is the proposed geological horizon for Canada’s Low and Intermediate Level Waste Repository. The Cobourg outcrops along the Northern shore of Lake Ontario and is also known as the Lindsay Formation. Samples from four locations were found in the literature, and the UCS and $E_i$ values are compared in Figure 3.19.
At first inspection there is wide scatter in the test results, with obvious outliers. The main outliers are from the Bowmanville quarry, and these may have been influenced by blasting. Detailed information relating to the sample collection at Bowmanville was not available, and therefore these points could not be screened further. Generally though, the trend and scatter is comparable from all four locations (Figure 3.19). The linear fit for all the data indicates that the MR for the Lindsay/Cobourg Formation is 368.

The data shown in Figure 3.19 originates from the near surface and at depth from the DGR. Higher strength samples are found from the DGR site than from other locations, but the lower bound of the strength range is similar. The stiffness range is comparable from all four locations, suggesting that there is little influence from the current or past burial depths. The stronger samples could be the result of better sample collection and transportation procedures or related to natural variability. For example, less argillaceous content may mean greater stiffness.

The natural variability of the Lindsay/Cobourg Formation influences the CI and CD thresholds, and there is a much larger range of CI and CD values for the Lindsay/Cobourg compared to the Queenston (Figure 3.20). The heterogeneous character of the Lindsay/Cobourg causes crack initiation at sites of contrasting stiffness earlier than within similar homogenous samples, and the argillaceous content absorbs propagating cracks resulting in higher CD values than the less argillaceous samples. In contrast, the Queenston is typically more homogenous, and so the damage thresholds have a narrower range.

As previously discussed, this is particularly true for CI. A comparison between samples from different locations for both the Lindsay/Cobourg and Queenston formations reveals that the wide and narrow ranges, respectively, are similar at different locations. This suggests that the ranges in these formations are applicable for preliminary design purposes for new projects.
Figure 3.20: Comparison of CI and CD values from different locations for the Lindsay/Cobourg Formation (top), and for the Queenston Formation (bottom), with data from Gorski et al (2009, 2010, 2011) and the present study.
3.4.3 Geomechanical Classification of Sedimentary Rocks

For geological engineering design, classification systems are used to determine zones of like behaviour based on Q (Barton 2002), RMR (Bieniawski 1989) or GSI (Hoek and Brown 1997) or other systems. These systems are typically mechanical behavioural classifications that focus on the rock mass. Typically the rock mass will be broken into different zones of behaviour.

In the case of mudrocks and carbonates there are several varieties of these broad rock types, as indicated by the respective geological classification systems of Lundegard and Samuels (1980) and Dunham (1962). It is common in geological engineering to simply refer to a shale or limestone. To address this disconnect between mechanical classification of rock masses and geological classification for rock types, a preliminary geomechanical classification system is proposed based on the laboratory testing results discussed in this chapter which are used to identify different mechanical behaviour of the classified rock types. The system will be able to distinguish between the different geological classes and give a preliminary idea of the expected geomechanical behaviour. The geomechanical classification system has three main components; the matrix, the grain size, and the behaviour. The classification is based on the grain size, which is used to derive the rock name and the mechanical behaviour describes generally the strength, stiffness, CI and CD characteristics (see Figure 3.21).

Generally speaking the more homogenous the rock is, the more uniform the mechanical behaviour will be. When there are varying grain sizes within a sample, CI and CD are largely controlled by the larger grain size components and depending on the relationship between the matrix and grains, these properties can vary considerably. The geomechanical system presented here is largely based on the rock properties from Ontario and should be expanded to include ranges of the physical properties based on rock testing results from around the world.
<table>
<thead>
<tr>
<th>Matrix Description</th>
<th>Classified Rock Name</th>
<th>Geomechanical Behaviour</th>
</tr>
</thead>
</table>
| Cement is commonly silicate or carbonate based and generally develops during diagenesis. Matrix minerals are grains < 0.03 mm and are commonly micas, quartz, feldspars or clays. | **Sandstone**  
Grains 0.06 – 2.0 mm | Cement controls magnitude of strength and stiffness, which is typically uniform at the excavation scale. Crack initiation and propagation influenced by cement and secondary mineralogy. Small variations in mineralogy have little influence on strength thresholds. |
| Depositional textures easily identified, depositional textures show diffusional identity | **Siltstone**  
> 67 % grains 0.004 – 0.06 mm | Lamination varieties include:  
- grain size segregation,  
- fabric or alignment of grains, and  
- colour variations. |
| **Non-Laminated**  
Matrix can consist of up to 33 % clay sized particles. Induration by consolidation or cementation. | **Mudstone**  
33 – 67 % grains < 0.004 mm  
**Mudshale** | Interbedding with different lamination types controls strength and stiffness. Small variations in mineralogy have little influence on strength thresholds and laminations influence propagation. |
| **Laminated**  
Distinction between matrix and grains can be difficult. Induration by consolidation. | **Claystone**  
< 33 % grains > 0.004 mm  
**Clayshale** | Clay content controls strength and stiffness. Stiffer grains cause crack initiation during loading and crack propagation is influenced by laminations. |
| Less than 10% grains greater than 20 μm  
Components not bound together during deposition | **Mudstone**  
Matrix grains < 5 mm | Strength and stiffness controlled by bedding, which are primary crack initiation sites |
| Mud supported with 10-30% grains by volume | **Wackestone**  
Grains < 30 mm  
**Floatstone**  
Grains > 30 mm | Strength and stiffness influenced by the size and shape of the large grains. Bedding is a primary crack initiation site, followed by grain boundaries. |
| Grain supported with mud filling voids. Greater than 20-30% grains by volume | **Packstone**  
Grains < 30 mm  
**Rudstone**  
Grains > 30 mm | Strength and stiffness controlled by grains and degree of mud infilling. Grain boundaries are the primary crack initiation site. |
| Grain supported without mud infilling the void space | **Grainstone**  
Grains < 30 mm  
**Rudstone**  
Grains > 30 mm | Strength and stiffness controlled by grain-grain contact, cement and grain integrity. Grain contacts are the primary crack initiation site. |
| Components bound together during deposition, | **Boundstone**  
Grains < 5 mm | Strength and stiffness controlled by binding organism and binding method. Variable crack initiation sites. |
| Depositional texture not recognizable | **Crystalline** | Strength and stiffness increase with increasing pervasiveness of dolomitization or re-crystallization. Pores spaces and flaws left behind from the conversion process are common crack initiation sites. |

Figure 3.21: Updated to the combined siliciclastic-carbonate classification system (Figure 2.11) with characteristic geomechanical behaviour for each class based on the present study.
3.5 Conclusion
In this study, sedimentary rock properties within Ontario were examined to determine both the range of values and how well the different test results correlate with one another, in addition to the degree of uniformity in the values at the formation level from different locations. In so doing, it was determined that the point load index is a reliable method of estimating the UCS, which can be used in conjunction with the MR to determine the intact rock modulus, $E_i$. Following the approach of Hoek and Diederichs (2006), this allows for an estimate of the rock mass modulus to be made for preliminary design purposes. The different rock classes, defined by Dunham’s (1962) system for carbonates and by Lungard’s and Samuels’ (1980) system for mudrocks, are a useful way of distinguishing rocks that behave in different manners mechanically, and using these classification systems allows for reasonable prediction of properties from one location to another. It was demonstrated that the mechanical behaviour of the Queenston and the Lindsay/Cobourg formations are fairly uniform at different locations. These findings demonstrate that it is possible to gain a good understanding of the potential range of sedimentary rock property values for assessment and preliminary design purposes. In addition, field testing can be used to discover which areas require further field investigations and laboratory studies in order to determine the required properties for detailed design purposes.

3.6 Acknowledgements
A special thank you is due to the Nuclear Waste Management Organization (NWMO) of Canada and Ontario Power Generation who provided data and access to samples for testing by the authors to supplement the existing data set gathered from the literature. NWMO and the Natural Sciences and Engineering Research Council of Canada have jointly sponsored this research through an Industrial Postgraduate Scholarship. A portion of the testing, not gathered from the literature, was
conducted at CANMET Mining in Ottawa, Ontario, and a portion at the Royal Military College of Canada in Kingston, Ontario. The assistance of Mr. Vanvolkingburgh and Mr. Gaskin is greatly acknowledged. This work was greatly enhanced by the assistance of Mr. Oke, from Queen’s University and the Royal Military College of Canada during equipment setup, calibration, and testing.

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Chapter 4: A Review of the Tensile Strength of Rock: Concepts and Testing

4.1 Abstract
A review of the tensile strength of rock was conducted to determine the relationship between Direct Tensile Strength (DTS) and Brazilian Tensile Strength (BTS) and to examine the validity of estimating tensile strength from other measured properties, such as the Crack Initiation Threshold (CI). A data set was gathered from the existing literature were tensile values could be reliably correlated with Unconfined Compressive Strength or Crack Initiation values. It was determined that the BTS obtained in standard testing is generally greater than the equivalent DTS and that this relationship is rock type dependent. Crack Initiation yields a reasonable estimate of tensile strength and this correlation is improved when the BTS values are reduced to DTS values by rock type specific correlations. The factor $f$, in DTS = $f$ BTS, can be considered to be approximately 0.9 for metamorphic, 0.8 for igneous and 0.7 for sedimentary rocks. The relationships presented demonstrate that there is wide scatter in the available data for estimating tensile strength likely due to both specimen variability and testing configuration, including platen geometry and relative stiffness. Estimates of tensile strength should only be used for preliminary design purposes and measurements should be made to confirm preliminary assumptions for each design.

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2 This chapter appears as submitted to an international journal with the following citation: Perras MA, Diederichs MS (2014) A review of the tensile strength of rock: Concept and testing. Geotechnical and Geological Engineering. Online first article. doi: 10.1007/s10706-014-9732-0
4.2 Introduction

Despite the importance of the tensile capacity in controlling many failure processes, tensile strength determination is often overlooked in engineering practice due to difficulties with obtaining reliable results. The initiation of fractures in brittle materials can be a tensile phenomenon as indicated by many researchers (Haimson and Cornet 2003; Myer et al. 1992; Stacey 1981; Taponnier and Brace 1976; Griffith 1921). Therefore the tensile strength is an important aspect of the resistance to failure of a rock or rock mass. Diederichs and Kaiser (1999) stated that tensile strength is an important controlling property in critical span stability of underground openings. Around underground excavations zones of low confinement develop as the stress field changes in response to the excavation and in some cases zones of tensile stress can develop. In these areas excavation damage develops in an extensile manner, particularly near the excavation boundary.

The focus of engineering studies is often on the unconfined compressive strength (UCS) of intact rock samples or confined compressive strength. This is partly due to the emphasis of confinement when determining the failure envelope for a rock mass (Hoek 1983) and partly due to difficulties in performing tensile tests. Direct tensile strength (DTS) testing is rarely carried out because of the difficulties in preparing the samples; many poorly-prepared samples fail invalidly (not through the middle of the sample) and thus must be discarded. Indirect tensile methods, such as the Brazilian Tensile Test, are much easier to prepare although invalid tests (fracture is not through the middle of the sample or visible platen effects) are also frequent and each sample should be examined after testing to determine its validity (Colback 1966).

An accurate and representative direct tensile strength value (DTS) forms the anchor for most rock strength envelopes defined in stress space. The Hoek-Brown (1997) formulation, for
example, originates at a tensile strength value \( T = -\text{DTS} = -\text{UCS}/m_i \), where \( m_i \) is a slope parameter for shear strength). Other methods may use a tensile cutoff limit in addition to a separate shear strength envelope. Given the widespread difficulty of determining direct tensile strength, reported in the literature (e.g. Berrenbaum and Brodie 1959; Mellor and Hawkes 1971), the authors reviewed methods for obtaining tensile strength indirectly. The most commonly used indirect method is the Brazilian Tensile Strength (BTS) test. The review focuses on the relationship between DTS and BTS for a variety of rock types found in the literature. Other methods of estimating tensile strength are also evaluated.

Tensile strength is also closely related to the stress threshold for fracture initiation in compression. This limit has been called the Crack Initiation limit or CI (Diederichs and Martin 2010). The estimation of crack damage thresholds (other than peak strength) using strain or acoustic emission based methods is becoming more common for routine UCS testing as it can be done with little additional effort during testing. Diederichs and Martin (2010) summarized the available methods for measurement of crack damage thresholds, which includes crack initiation (CI) and crack propagation (CD). These values are important parameters for input into numerical models. In particular the damage initiation and spalling limit (DISL) approach of Diederichs (2007) requires the input of UCS, CI, and tensile strength to describe the rock mass failure envelop. The DISL approach is a method of determining the peak and residual failure envelopes to capture the brittle behaviour of rock masses around underground excavations. UCS and CI values are more commonly available or measured; however, tensile strength is often overlooked. The relationship between tensile strength, crack initiation and excavation damage will be explored further.
4.3 Measuring Tensile Strength in the Laboratory

Measuring the tensile strength of rock is governed by the ISRM (1978) standard which outlines both direct and indirect Brazilian test methods. ASTM also has standards governing direct (ASTM 2008a) and indirect Brazilian (ASTM 2008b) methods. These standards specify, for example, cylindrical samples with Height to Diameter ratios between 2.5 to 3.0 with cemented end caps for direct tests, and cylindrical discs with a Radius to Thickness ratio of 1.0 for Brazilian tests. As these are the standard tensile test methods they are discussed in detail as well as variations on these methods, which have been presented in the literature. For completeness other indirect methods are discussed briefly.

4.3.1 Direct Tensile Strength

To date direct tensile testing is regarded as the most valid method for determining the true tensile strength of rock since there are minimal outside influences when the test is completed properly (Hoek 1964). Brace (1964) described the best shape for direct tensile samples to be the dog bone shape, as illustrated in Figure 4.1a, where the Height, H, to Diameter, D, ratio should be 2.0 to 3.0 of the central test region. The Radius, R, of curvature of the fillets should be approximately 1 to 2 x D. Samples of this shape can be tested with grips, as illustrated in Figure 4.1a, which pull against the lip of the dog bone ends or simply grip the vertical edge of the sample, either mechanically or with adhesive. It should be noted that it is the curved radius of the dog bone shape which reduces stress concentrations at the ends of the sample. In this review the authors have observed that square dog bone samples are also sometimes used. The square shape can still result in stress concentrations at the ends of the sample during loading and invalid failure away from the central area of the sample can still occur.
Figure 4.1: Direct tension testing arrangements using a) split grips for dog-bone shaped samples (shape described by Hoek 1964), b) glued end caps for cylindrical samples (ISRM 1978), c) biaxial extension after Brace (1964) and, d) compression to tension load converter, for tensile testing after Gorski (1993) and Klanphumeesri (2010). $F_a =$ applied force, $R =$ radius, $D =$ diameter, $A =$ area, and $P =$ water pressure.

Gripping a cylindrical sample (non-dog bone) sample can cause stress concentrations at the ends near the grips. A valid direct tensile test should result in failure at the midpoint of the sample. With stress concentrations at the ends of the sample failure can initiate near the grips resulting in an invalid test.

In work by Fairhurst (1961) the stress concentrations were reduced by directly cementing end caps of the same diameter to the sample ends (Figure 4.1b), which results in a similar stress
distribution as in a uniaxial compression test. The difficulty with the fixed grips or end caps is that the sample can fail by bending if the end caps are not aligned exactly. Even a minor offset could a moment and failure of the sample will not be in pure tension. This is mostly overcome by using universal or ball joints to allow the sample to centre itself during loading. These difficulties are avoided in biaxial extension tests since end grips are eliminated from the experimental procedure.

4.3.1.1 Biaxial Extension
Hoek (1964) and Brace (1964) were able to make the axial stress tensile using dog-bone shaped samples in a triaxial cell, where \( \sigma_1 = \sigma_2 > \sigma_3 \). Jaeger and Cook (1969) state that in practice both \( \sigma_1 \) and \( \sigma_3 \) are increased together to confine the sample and that the axial stress is then decreased until failure occurs. Utilizing the dog bone shape, \( \sigma_3 \) then exerts a pressure on the curved portion of the sample and causes extension. The test setup is illustrated in Figure 4.1c. The tensile strength can be calculated using Equation (4.1);

\[
T = \frac{F_a}{A_1} - \frac{P(A_2-A_1)}{A_1}
\]  

(4.1)

where \( F_a \) is the applied axial load, \( A_1 \) is the narrow diameter, \( A_2 \) is the head diameter, and \( P \) is the confining pressure. It should be noted that biaxial extension is not a purely uniaxial tensile test; although Brace (1964) states that the dog-bone shape sample yields more reliable data over other tensile methods. If no axial force (\( F_a \)) is applied than the arrangement becomes a uniaxial tensile test (Hoek 1965). In addition it is important that the data are plotted on the appropriate stress path, which in turn is a function of the curvature of the fillet.

In many laboratories tensile (pull) loading frames or confining cells are unavailable. Research has been conducted by others (Gorski 1993; Klanphumeesri 2010) to utilize
compression load frames to convert the downward compression into an upward pull on dog-bone shaped samples.

4.3.1.2 Compression Load Converters
Gorksi (1993) patented a device which allows the conversion of the compressive load to pull a dog bone shaped sample apart in tension. This device can incorporate the curved dog bone shape recommended by Brace (1964) and Hoek (1964). A study by Klanphumeesri (2010) demonstrated the use of a similar load converter for direct tensile testing. The device used by Klanphumeesri (2010), illustrated in Figure 4.1d, is simpler to construct than the device of Gorski (1993) and sample installation is easier. Even small time savings during sample preparation and installation into the loading frame can increase the number of samples which a laboratory can test within practical time constraints. Lathing samples to achieve the desired testing shape remains the limiting factor.

In all cases of direct tensile testing the tensile strength can be directly measured as the load at failure, $F$, divided by the cross sectional area, $A$, of the failure zone, with the exception of biaxial extension. This is the ideal method of determining the tensile strength of rock, however, preparation of the samples can be time consuming and difficult, especially with weak rocks. In some cases, such as shale, the sample will be unable to withstand shaping on a lathe to create the recommended dog bone shape. Due to the large variability of tensile strength of rock samples, even of the same rock type, many tests are recommended (Jaeger and Cook 1969). To meet this recommendation it is most practical, in terms of preparation time, to use grips or glued end caps on straight samples or to switch to an indirect testing method.
4.3.2 Brazilian Tensile Strength

The Brazilian Tensile Strength (BTS) is determined by an indirect testing method governed by the ISRM (1978) and ASTM (2008b) standards, which state that the stress at failure, $T$, is a function of the applied load $F_a$, the diameter $D$ and the thickness $t$ at the centre of the sample and can be determined by Equation (4.2).

$$T = \frac{2F_a}{\pi D t} = 0.636 \frac{F_a}{D t}$$  \hspace{1cm} (4.2)

Li and Wong (2012) documented the development of this method starting with the independent proposals for concrete by Carneiro (1943) and Akazawa (1943). The first application for rock was by Berrenbaum and Brodie (1959). Recent studies focus on applications and improvements through both laboratory and numerical studies (Tavallali and Vervoort 2010; Markides et al. 2012; Erarslan and Williams 2012; Li and Wong 2012). Many studies (see Li and Wong 2012) have focused on the loading platens and contact area for BTS testing to determine the optimum load transfer to the sample such that tensile stresses develop evenly in the central region and minimize crushing at the edge of the sample. The ISRM (1978) standard suggests a curved jaw with a radius of 1.5 times the sample radius; however, a variety of other loading platen arrangements are in use for the BTS tests and some of the most common are shown in Figure 4.2.

Hondros (1959) accounted for a load of finite width on a solid disk, with the stress components given by Equations (4.3) and (4.4), modified from Mellor and Hawkes (1971), such that tension is negative.

$$\sigma_1 = -\frac{F_a}{\pi R t a} \left\{ \left[ \frac{1-(\frac{r}{R})^2}{1-2(\frac{r}{R})^2 \cos 2\alpha + (\frac{r}{R})^4} \right] \sin 2\alpha - \tan^{-1} \left[ \frac{1+(\frac{r}{R})^2}{1-(\frac{r}{R})^2} \tan a \right] \right\}$$  \hspace{1cm} (4.3)

$$\sigma_3 = +\frac{F_a}{\pi R t a} \left\{ \left[ \frac{1-(\frac{r}{R})^2}{1-2(\frac{r}{R})^2 \cos 2\alpha + (\frac{r}{R})^4} \right] \sin 2\alpha + \tan^{-1} \left[ \frac{1+(\frac{r}{R})^2}{1-(\frac{r}{R})^2} \tan a \right] \right\}$$  \hspace{1cm} (4.4)
where $F_a$ is the applied load, $R$ is the disc radius, $t$ is the disc thickness, $2\alpha$ is the angular width where the load is applied over (radially), and $r$ is the distance from the centre of the disc. $\sigma_1$ is the vertical stress within the sample and $\sigma_3$ is the horizontal stress within the sample during loading.

The sensitivity of the normalized stress to the angular width is demonstrated in Figure 4.3a, where an increasing angular width decreases the normalized tensile stress at the centre of the disk.

A closer examination of the tensile stresses (Figure 4.3b) shows that there is approximately an error of 10% between a point load and a width of $2\alpha = 30$. Also, as the angular width increases the distance ($r$) over which the tensile stress is constant decreases. Theoretically a point load will give the lowest tensile strength; however, in practice a point load will cause crushing at the point of contact with the sample.

Figure 4.2: Typical loading platen arrangements for BTS testing using a) flat loading platens, b) flat platens with cushion (often wood), c) flat loading platens with small diameter rods, or d) curved loading jaws (ISRM 1978). $F_a =$ applied force and $D =$ diameter.
Figure 4.3: a) Distribution of normalized stresses with b) a close up view of the tensile (-) region and c) the stress difference along the loading diameter (r/R) of a solid disc of unit thickness (t) for different loading widths (2α), after Hondros (1959). R = radius of sample, r = distance from centre of sample, P = force, σ_{vert} = vertical stress in the sample (σ₁), σ_{horiz} = horizontal stress in the sample (σ₃), CI = crack initiation, and σ_t = tensile strength.
Examining the stress difference (Figure 4.3c) along the loading axis shows that it is greatest just inside the sample near the platen and the magnitude of the difference increases as the angular width decreases. Further from the edge of the sample, in the central region, the stress difference becomes similar, independent of the loading width. Griffith’s (1921, 1924) theory and the expanded three dimensional version suggested by Murrel (1963) can be used to explain the limiting loading angle width \(2\alpha\) to minimize crack initiation of the sample in the platen area.

The equations relating the principal stresses \((\sigma_1, \sigma_3)\) and tensile strength \((T)\) in Equation (4.5) and Equation (4.6) are as follows:

\[
\sigma_1 = \frac{-8T(1+\frac{\sigma_3}{\sigma_1})}{(1-\frac{\sigma_3}{\sigma_1})^2} \tag{4.5}
\]

\[
\sigma_1 = \frac{-12T\left(1+2\frac{\sigma_3}{\sigma_1}\right)}{(1-\frac{\sigma_3}{\sigma_1})^2} \tag{4.6}
\]

after Griffith (1921, 1924) and Murrell (1963), respectively. Diederichs (1999) reiterated that Griffith’s theory is a damage initiation threshold over the full range of confining stress and that while in compression, crack accumulation or propagation is required (after initiation) to fully fail a crystalline rock sample. In tension the damage initiation and peak strength are coincident due to unstable crack propagation. In an unconfined test \((\sigma_3 = 0)\) these equations reduce to \(\sigma_1 = -8T\) and \(\sigma_1 = -12T\), respectively. In a DTS test CI and peak tensile strength should occur almost simultaneously and therefore \(\sigma_1 = CI\). Following from this crack initiation can occur in the platen area when \(2\alpha < 16\) since the stress difference will exceed \(CI = 8T\) or \(CI = 12T\), according to Griffith (1921, 1924) or Murrell (1963), respectively, as indicated in Figure 4.3c.

Even with a small distributed load during BTS testing, crushing in the platen area can occur for weak samples. The authors have observed that laboratories use a cushion, such as plywood or cardboard, to prevent contact crushing in the platen area of weak samples. It should
be noted however a cushion that spreads out during loading can also prematurely crack the sample in tension. For the Brazilian Tensile test on a sample to be valid the fracture should start and pass from the central region of the sample out towards the loading platens. Invalid tests often occur in metamorphic or sedimentary samples; where the fractures deviate along the fabric plane or when fracturing begins in the platen area. Mixed failure modes can also occur where fractures propagate to the platens and other locations.

A variation of the Brazilian test, originally developed to overcome a tendency for mixed failure modes in solid disks, is the Ring Test. The tensile strength of a ring test can be determined by Equation (4.7),

\[ T = \frac{2F_aK_f}{\pi D t} \]  

(4.7)

where \( F_a \) is the applied load, \( D \) is the disk diameter, \( t \) is the disk thickness and \( K_f \) is the stress concentration factor. This factor can be calculated using Equation (4.8).

\[ K_f = 6 + 38 \left( \frac{r}{R} \right)^2 \]  

(4.8)

Where \( r \) is the hole radius and \( R \) is the disk radius. This type of test is now infrequently used; however other tensile tests are still in use or are being more recently developed to overcome the difficulties with direct and indirect methods.

### 4.3.3 Alternative Tensile Testing Methods

An exhaustive review of other tensile tests is beyond the scope of this paper. Vutukuri et al. (1974) give a thorough review of early methods used for tensile strength determination. Several of the best studied tests, including those more recently investigated (Luong 1990) are illustrated in Figure 4.4, which shows the relationships between the test parameters and the test specific tensile strength. The sleeve-fracturing test is a variation of a dilatometer test which is used in-situ to determine the deformability of a rock mass, where an internal pressure is applied until the
sample splits radially (Figure 4.4a). There are many different types of beam bending tests used in practice, most of which are a variation of a three or four point bending test, as illustrated in Figure 4.4b, with the associated governing equations for tensile strength determination. The most recent development in tensile strength testing is the modified tension test developed by Luong (1990) in which a tensile zone develops during loading of an over cored sample in the rock bridge, as illustrated in Figure 4.4c.

Despite extensive testing with alternative tensile testing methods for research purposes, in practice the tensile strength is in some cases estimated from other index test values. The rest of this paper will focus on BTS as it is routinely used in industry for tensile strength determination. As DTS is considered to be measure the true tensile strength of rock, this paper will compare test results from the literature with BTS. Since there is a large volume of BTS test results in the literature, these are compared with other routine testing results or estimates. It will be shown that reasonable estimates of the true tensile strength of rock can be determined by reducing BTS values or estimated using CI.

4.4 Estimating Tensile Strength

Rock properties are often correlated between field measurements and laboratory test results, such that in similar rock masses preliminary engineering design can be conducted prior to obtaining laboratory test results. It should be stressed that these relationships are for preliminary analysis only and should never replace laboratory testing to determine tensile strength.

Many authors have made correlations between UCS and the point load index, \( I_{50} \), as summarized by Zhang (2005). Zhang (2005) suggests a correlation between tensile strength and the point load index and reports the relation in Equation (4.9).

\[
T = -1.5 I_{50}^{(50)}
\]  

(4.9)
Figure 4.4: Alternative indirect tensile testing methods, including a) the sleeve fracturing test, b) the beam bending test, and c) the modified tension test, as described by Franklin and Dusseault (1989) and Luong (1990). $P =$ water pressure, $r =$ inner radius ($r_2$), $R =$ outer radius ($r_1$), $T =$ tensile strength, $F_a =$ applied force, $L =$ length, $h =$ height, and $b =$ thickness into the page.
Hoek and Brown (1997) established the material constant, $m_i$, to describe the relationship between the principal stresses and the peak strength, as shown in Equation (4.10), for intact rock.

$$\sigma_1 = \sigma_3 + UCS \left( m_i \frac{\sigma_3}{UCS} + 1 \right)^{0.5}$$  \hspace{1cm} (4.10)

It is possible, but not recommended, to fit only the triaxial and UCS data, and then estimate the tensile strength by calculating $m_i$ (Hoek and Brown 1997). If only UCS testing has been completed and reliable tensile testing data are unavailable, an estimate can be made using $T = -\frac{UCS}{m_i}$, for brittle rocks (Diederichs 2007) and determining $m_i$ based on Hoek and Brown’s (1997) recommendations according to the rock type.

As previously discussed, the tensile strength can be related to the CI threshold. Standard UCS testing is typically the first stage of testing on any engineering project and therefore the tensile strength could be estimated using CI. If $\sigma_3$ is set to zero in Equation 4.5 than $\sigma_1 = 8T$. In tension the CI threshold is equivalent to the peak strength, so $CI = 8T$. Similarly, if Equation 4.6 is used, $CI = 12T$. The range (8 to 12) in the relationship between tensile strength and CI can be generalized into Equation (4.11);

$$T = \frac{CI}{\beta}$$ \hspace{1cm} (4.11)

where $\beta$ is 8 according to the original Griffith (1921, 1924) theory and can be as high as 12 according to the modified formulation of Murrell (1963) and Jaeger and Cook (1969).

These estimates should only be used as preliminary design estimates for tensile strength and are no substitute for quality testing on intact rock samples for tensile strength. As has already been mentioned, the difficulties with direct tensile testing often mean that indirect methods are used to determine the tensile strength. A search of the literature, summarized in Table 4.1, was conducted to examine the relationships between standard properties measured in the lab and between direct and indirect (Brazilian) tensile testing methods.
Table 4.1: Summary of average filtered rock properties found in the literature with the number of samples indicated in brackets. Values are for results which can be reliably correlated between tensile strength (DTS and BTS) and the other strength thresholds (UCS, CI, and CD). Note that the individual values are used in the figures and not the average values.

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>DTS (MPa)</th>
<th>BTS (MPa)</th>
<th>UCS (MPa)</th>
<th>CI (MPa)</th>
<th>CD (MPa)</th>
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<tr>
<td>Amphibolite</td>
<td>-</td>
<td>8.9 (2)</td>
<td>110.0 (1)</td>
<td>64.0 (1)</td>
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<td>Aplite</td>
<td>-</td>
<td>-</td>
<td>309.9 (5)</td>
<td>151.4 (5)</td>
<td>262.9 (5)</td>
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<tr>
<td>Basalt</td>
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<td>63.1 (1)</td>
<td>-</td>
<td>-</td>
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<td>Calcarenite</td>
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<td>0.6 (1)</td>
<td>-</td>
<td>-</td>
<td>-</td>
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</tr>
<tr>
<td>Chert</td>
<td>34.8 (1)</td>
<td>-</td>
<td>610.9 (1)</td>
<td>-</td>
<td>-</td>
<td>Courtesy of Dr. Evert Hoek</td>
</tr>
<tr>
<td>Diorite</td>
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<td>251.7 (10)</td>
<td>117.5 (10)</td>
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<td>8.0 (6)</td>
<td>90.4 (4)</td>
<td>35.6 (4)</td>
<td>75.3 (4)</td>
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<th>CI (MPa)</th>
<th>CD (MPa)</th>
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<td>148.3 (5)</td>
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<td>-</td>
<td>-</td>
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<td>101.6 (48)</td>
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<td>-</td>
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<td>Jacobsson 2005, 2006, Cai 2010</td>
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<td>-</td>
<td>-</td>
<td>Jaeger 1967, Graue et al. 2011</td>
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4.5 Correlations with Tensile Strength

Coviello et al. (2005) compiled various alternative test results from the literature which were compared to DTS. These have been re-plotted in Figure 4.5, along with mean testing results from Coviello et al. (2005). The results indicate that the BTS and the Hydraulic Fracture (results processed using elasto-plasticity shown in Figure 4.5) tests give the closest approximation to the DTS. Given the large volume of BTS strength data in the literature, as evident in Figure 4.5 and in Table 4.1, the focus of this paper is largely on the relationship between DTS and BTS.

Estimation of DTS using CI and the point load index is also examined as a preliminary method when tensile test results are unavailable.

Despite the large collection of laboratory testing results in the literature it is very difficult to determine whether different data sets use the same rock block or core run interval. In order to correlate compressive and tensile laboratory results it is important that the samples came from very close to each other as natural variation should be minimized. The samples which were compared by the original authors were assumed to have originated from close special proximity to each other.
Figure 4.5: Comparison between DTS and various alternative indirect tensile testing methods, where 3PB is 3 point bending, 4PB is 4 point bending. Alternative tensile test data compiled from Coviello et al. 2005, with data from Hardy and Jayaraman 1970, Jaeger 1967, Jaeger and Hoskins 1966, and mean values from Pandey and Singh 1986 and Klanphumeesri 2010. For complete references to data for BTS and DTS only, see Table 8.1.
Where a comparison was not made by the original authors, but spatial information was available, samples from within 10 cm were compared. Still there is wide scatter in the relationships presented here.

### 4.5.1 Crack Initiation under Compressive Loading

CI represents the first onset of new distributed grain scale cracks within the sample during testing. Tapponier and Brace (1976) found that natural micro-cracks below the CI threshold are generally limited to the grain scale for crystalline rocks. As noted previously, the crack initiation threshold under compression is a robust material property and is mechanistically related to the direct tensile strength (Diederichs 2003). In contrast, the threshold for critical crack accumulation, propagation and interaction, CD is more associated with loading conditions and confining stress. UCS, in turn, is thereafter related to testing geometry, loading rate and other influences in addition to the rock properties.

The different methods for determining the damage thresholds are illustrated in Figure 4.6 and are only briefly discussed here for clarity. The CI threshold can be determined as the point where the stress-strain (volumetric or lateral) deviates from linearity (Brace et al. 1966; Bieniawski 1967; Lajtai 1974). If there is pre-existing damage (micro-cracks) in the sample then the linear elastic behaviour may not be present or the true onset of crack initiation may not be accurately defined. Crouch (1970) suggested that the lateral strain-axial strain plot could be used to determine CI. However, around CI, during a compression test, both lateral and axial strains can change in a similar fashion and therefore mask the true CI threshold because of the delay in the deviation from linearity. The CI threshold can alternatively be determined as the deviation from linearity of the Inverse Tangent Lateral Stiffness (ITLS) versus applied stress plot, after Ghazvinian et al. (2011). The ITLS method avoids the problems stated above by examining the
rate of change in the slope itself. The CD threshold is the point where cracks begin to occur in an unstable manner (Bieniawski 1967) and the cracks begin to interact. CD can be determined, in an unconfined compression test, as the deviation from the linear elastic response of the strain in the direction of loading. CI and CD thresholds can also be detected using acoustic emission (AE) sensors. Changes in the rate of acoustic emissions during testing have been found to correlate well with the damage thresholds discussed above (e.g. Scholtz 1968; Lockner 1993; Falls 1993; Eberhardt et al. 1998), as shown in Figure 4.6.

Figure 4.6: Different methods of determining Crack Initiation (CI) and crack propagation (CD) thresholds during a compression test. CC = Crack Closure, Vol. = volumetric, ITLS = Inverse Tangent Lateral Stiffness, and AE = Acoustic Emission.
The authors conducted a series of numerical analyses to determine the sensitivity of the DISL approach to the selected tensile strength used as an input. The mean values of a set of UCS, CI and BTS test results for a limestone were used and a variety of maximum tangential stress \( (\sigma_{\text{max}}) \) levels.

Tensile strengths were also estimated by reduction of the mean BTS value and using CI/8 and CI/12 (to be discussed later in the paper). The tensile strength decreases in the following order; mean BTS, reduced BTS, CI/8, and CI/12. The stress evolution at the notch tip is such that the rock mass failure envelope is crossed with a positive \( \sigma_3 \). With all other parameters being equal, decreasing the tensile strength causes an increase in the slope of the failure envelope for positive \( \sigma_3 \) values.

The study examined the change in the Excavation Damage Zones (EDZs) with different tensile strengths. The EDZs were determined for each model, following the methodology of Perras et al. (2012). The outer EDZ (EDZo) was determined as the maximum limit of the plastic yield zone. The inner EDZ (EDZi) was determined as the volumetric extension-compression transition. The Highly Damaged Zone was determined as a low confinement area with rapidly increasing volumetric and maximum shear strain toward the excavation boundary. The results of the study are shown in Figure 4.7, which indicates that there can be up to a difference of 20% of the predicted damage zone dimension. The increased slope with lower tensile strength (e.g. CI/12) values actually causes the damage zone dimensions to be smaller than when using a higher tensile strength (e.g. mean BTS). This could be a difference of between 0.5 to 1.0 m for a typical access shaft.

For nuclear waste disposal underground, the damage zone dimensions need to be predicted so that mitigation measures can be adequately designed. The typical mitigation measure
envisioned is to create a slot perpendicular to the excavation axis which cuts off flow along the EDZ (flow parallel to the excavation axis). If the slot is too shallow, the cut off slot will be ineffective and if it is too deep there is a risk of causing more damage to the rock mass, which may create a short circuit at the cut off slot tip. This numerical study raises the question what is the most appropriate tensile strength value to include in an analysis and whether there are reliable methods of estimating tensile strength when data is unavailable.

Figure 4.7: The influence of different tensile strengths on the dimensions of the outer Excavation Damage Zone (EDZo), the inner Excavation Damage Zone (EDZi) and the Highly Damage Zone (HDZ), where BTS is the mean Brazilian Tensile Strength and fBTS is the reduce Brazilian Tensile Strength determined from the current study.
4.5.2 Relationships between Intact Rock Properties

In an effort to determine the best relationship between direct tensile strength and other more commonly determined laboratory parameters the authors started with UCS. The comparison between UCS and BTS or DTS data shows (Figure 4.8) wide scatter, as previously mentioned. This is particularly true for the indirect Brazilian method (BTS). There is a better correlation with DTS although there is a similar range of variability and fewer data in the literature. The mean $m_i$ range for different rock types, 4 to 32 (Hoek and Brown 1997), should bracket the data.

![Figure 4.8: The relationship between UCS - BTS and UCS - DTS.](image)
Examining Figure 4.8 the ratio \( m_i = \frac{UCS}{T} \) does bracket the minimum and maximum limits of the data, approximately. When the information is included in the source publications, and the data can be separated by rock type, there is good agreement with the ratio UCS / T for different \( m_i \) values (Figure 4.9). If the data are not separated by rock type and study, then there is less alignment with different \( m_i \) values, which can be seen by the variation in the granite data points in Figure 4.9. This is because of the natural variation in granites or other rock types from different locations, such as grain size or mineralogical variations, for example.

CI and CD can also be determined from a UCS test by using strain measurements and/or acoustic emission sensors following the methods described previously. CI is an important parameter in the Damage Initiation and Spalling Limit method established by Diederichs (2007) and critical for understanding the behaviour of brittle rocks.

There is a well-established correlation between CI and UCS demonstrated by the work of Brace et al. (1966), which ranges between \( 0.3 < \frac{CI}{UCS} < 0.5 \). The data collected as part of this study is within this range (between a ratio of 0.4 for CI and 0.8 for CD as seen in Figure 4.10a). Segregating the CI values by main rock types shows that there is close agreement between the CI ratio for sedimentary and igneous rocks, with ratio of 0.42 and 0.43, respectively (Figure 4.10b). The metamorphic results indicate a slightly higher ratio of 0.45. CI is more closely related to tensile strength than CD since it is the point in a UCS or triaxial test where cracks begin to develop in the sample.

It should be noted that this data set is not a complete collection of UCS, CI, and CD measurements from the literature and is based only on measurements for which corresponding tensile strength data was available. Based on the available data for the igneous samples, a linear correlation which passes through the origin does not fit the data. The trend is left to illustrate that
the homogeneity of the igneous samples influences the behaviour and generates a more consistent CI value than metamorphic or sedimentary rocks.

Figure 4.9: The relationship between UCS – BTS and UCS – DTS by rock type and study.
Figure 4.10: The relationship between UCS and CI for a) the main data set and b) the main rock types; metamorphic, sedimentary and igneous. The subscript meta is for metamorphic, sed for sedimentary, and ig for igneous.
There are very few cases for which the spatial criteria were met between UCS or CI and DTS (only 6 tests for CI vs DTS, not shown). The importance and use of CI has only more recently been established and as such its routine measurement is still absent in many laboratories. There are 104 points, mostly from nuclear waste investigation programs, where stress-strain plots are available and CI can be determined from the published result or in some cases CI was calculated as part of the testing program. A comparison between BTS and CI indicates a general trend where BTS = 0.14 CI with a weak correlation (R² = 0.11), as shown in Figure 4.11.

Figure 4.11: Relationship between CI and BTS with Griffith’s (1921, 1924) 2D (CI/8) and Murrell’s (1963) 3D (CI/12) limits based on the main rock types.
Despite the weak correlation coefficient it is interesting to observe that the relationship closely corresponds to the tensile strength estimated by Equation (4.11) for a dense lower bound (Griffith’s (1921, 1924) $T = CI / 8$). A large portion of the data set is for metamorphic rocks, as shown in Figure 4.11, which shows a large variation. There is insufficient data for sedimentary and igneous rocks to establish specific trends. Figure 4.12 indicates that the sedimentary rocks are all dolomite and that the igneous rocks are either granite or granodiorite.

![Graph of CI vs BTS for specific rocks](image)

**Figure 4.12**: The relationship between CI and BTS for specific rocks. Note that metamorphic rocks were determined from the literature (reported or photograph).
The test results for the sedimentary rocks, in Figure 4.12, are unpublished and came from within the same borehole, although from different formations suggesting that there is a steep slope to the dolomite CI vs. BTS relationship. For the igneous rocks, the narrow window of CI values results from the fact that the data all comes from Finland, for CI vs. BTS, and is either granodiorite or granite, as shown in Figure 4.12 (Heikkilla and Hakala 1998; Eloranta and Hakala 1999).

The similarity in grain type and size may result in similar CI values, similar to the UCS relationship, suggesting that for homogenous rocks, CI is a material property. This concept is further reinforced on closer inspection of the individual rocks in Figure 4.12, which show alignment with common CI values for potentially more homogenous or uniform samples at the testing scale, such as schist and massive sulphide. Other rocks which are typically considered heterogeneous by nature show more scatter, such as sulphide mix, pegmatite, and meta-granite. Further testing is needed to confirm this finding.

Tensile strength determined from direct and indirect methods are seldom equivalent. Direct tensile testing is considered to yield the true tensile strength of intact rock whereas the Brazilian tensile test is considered to overestimate the true tensile strength of intact rock. A general rule of thumb used in practice is that DTS = 0.67 BTS or two thirds. If the complete data set for DTS and BTS values is considered, the linear regression yields the general rule of thumb closely (DTS = 0.65 BTS) with a coefficient of determination of 0.42, as shown in Figure 4.13a. The small red squares in Figure 4.13a are a series of tests reported by Gorski et al. (2007) on meta-granitoid samples from the Forsmark Nuclear Waste storage investigations in Sweden.

Orientation of fabric for the collected data is seldom indicated, however; a degree of foliation can be implied by the specific rock type. The DTS and BTS data, in Figure 4.14a, was
first broken down into specific rock types, which shows that for a highly foliated rock such as schist that there is a uniform one to one relationship between BTS and DTS. Other rock types which could be considered to be less foliated than a schist, such as sandstone and marble, show less steep trends. The granite points are variable and speculatively dependent on grain size and variations in mineralogy, although information is lacking in the literature in this regard. To examine this further, the data were ranked according to the degree of foliation as follows:

- No Foliation means there is no distinct orientation of minerals or separation implied or indicated.
- Weak means that there is a weak alignment of minerals without continuous bands.
- Moderate means that there is an alignment of mineral and separation of like minerals forming discontinuous bands.
- Foliated means that there is an alignment of minerals and separation of like minerals forming continuous and distinct bands.

Figure 8.14b shows that foliated samples have the closest relationship between BTS and DTS, where DTS = 0.96 BTS, followed by moderate and non-foliated with DTS = 0.83 BTS and that weakly foliated samples have the lowest relationship with DTS = 0.70 BTS. These relationships are very similar to the main rock type relations, which appear to capture the degree of foliation adequately, with metamorphic rocks being closest to a one to one relationship between BTS and DTS.
Figure 4.13: The relationship between BTS and DTS for a) the unfiltered and filtered data and b) for the filtered data with the main rock types. Note that the data were filtered based on the sample geometry. The subscript Filtered means the linear fit is through the filtered data only (diamonds) and All means the linear fit is through all the data (diamonds & squares). The subscript meta is for metamorphic, sed for sedimentary, and ig for igneous.
Figure 4.14: The relationship between BTS and DTS for a) specific rocks and b) for the degree of foliation. The subscript F = foliation, M = moderate foliation, W = weak foliation, and NF = no foliation.
As previously discussed the relationship between CI and BTS (Figure 4.11) has a weak coefficient of determination of 0.11. If the BTS values are reduced using $DTS = 0.85 \times BTS$, the linear relation for all the data, there is a slight change in the relationship, where $DTS = 0.12 \times CI$ (Figure 4.15a). There is no change in the $R^2$ since the reduction is simply linear.

However; if the main rock type relationships between BTS and DTS are used from Figure 4.13 then there is a slight increase in the coefficient of determination to 0.22, (Figure 4.15b). The relationship than becomes $DTS = 0.13 \times CI$, which corresponds to Equation 4.10 with $\beta = 8$.

Field tests are a desirable way to quickly assess the suitability of a rock mass for construction purposes. The point load index can give a quick estimate for preliminary design purposes. A comparison between the tensile strength estimated by the point load method (using Equation 4.9) and BTS results are shown in Figure 4.16. The normal fits to both methods agree very well, with the mean Point Load and BTS values being 6.3 and 6.0 MPa, respectively. The coefficient of variation (COV) is within the acceptable levels for geomaterials, according to Langford (2013). There are many influences which can contribute to the variability of the results.

4.5.3 Variability in Test Results

Three main sources of variability within the data presented include spatial relationships between samples compared, testing quality control and procedures, and natural heterogeneity. The authors used two criteria to compare the results of different test methods to ensure that the results are representative of the same rock type. Firstly the authors looked at testing results which had direct comparisons published by the original researchers. Secondly, where information about the spatial location, such as borehole number and depth, of the samples was provided by the original researchers, samples that were within 10 cm of each other were considered to be representative of the same rock.
Figure 4.15: CI compared to DTS calculated from the BTS test results using the linear relation of $DTS = f \cdot BTS$ where a) generally $f = 0.84$ and b) specifically $f = 0.93$ for metamorphic, 0.86 for igneous, and 0.68 for sedimentary rocks.
A distance of 10 cm, in drill core for example, is in the approximate range that one would be able to find both a UCS and a DTS or BTS sample that are similar in nature. In most cases spatial correlations in this manner were confirmed by examination of photographic evidence provided in the published testing reports, such as in Jacobsson’s (2004 through 2007) work.

A large volume of data came from testing for nuclear waste storage underground and testing results with different methods were often published in different reports. In all the collected data, the results of different test methods were only compared when the same author(s) published
the compressive and tensile results or if it could be confirmed that the same laboratory was used for the different test methods (as well as meeting the spatial criteria outlined at the start of this section). This should ensure consistent care in sample preparation and testing procedures.

However, the BTS loading platen could influence the final tensile strength. Mellor and Hawkes (1971) studied the influence and determined that a curved loading jig and a cushion gave reliable and consistent results.

To understand natural variability in the results, a sub-section of the collected data, granites, is used to discuss possible natural influencing factors. Natural variability, such as grain size variation, within a sub-set based on a specific rock type can account for some of the scatter in the data presented in this paper.

Due to limited amounts of specific rock type data, it is difficult to determine relationships between the test results. It is however possible to demonstrate that there is an influence based on where the samples have come from, as shown in Figure 4.17.

This graph presents data from a collection of granite and meta-granite samples from various locations. In Figure 4.17a, it can be seen that there are generally clusters or alignment of data points based on the location from which the samples originated, suggesting that the relationship between BTS and DTS is influenced by the nature of the specific granitic properties, such as grain size or mineralogical percentages. On average, per location, the result would plot very close to the $DTS = 0.84 \times BTS$ line, with the exception of the AECL – 420 level samples.

According to Gorski and Yu (1991) the AECL – 420 level samples came from vertical boreholes drilled from within an adit at the Pinawa Underground Research Laboratory in Manitoba. They concluded that the samples must contain micro-fractures from stress relief during drilling.
Figure 4.17: The relationship between a) BTS and DTS for granites from different locations and b) CI and DTS = f/BTS for meta-granites from Sweden and Finland.
The micro-fractures would be oriented perpendicular to the direction of drilling and this would reduce the DTS, since the sample would also be loaded perpendicular to the micro-fractures. The BTS would be less influenced by the micro-fractures since the direction of loading would be parallel to the micro-fractures (see inset diagram in Figure 4.17a). Lim and Martin (2010) analyzed core samples from the 420 level and showed that intact samples can be partially disked or contain micro-fractures.

For the relationship between CI and DTS (f BTS) clustering occurs based on where the samples originated from, although less clearly (Figure 4.17b). The Forsmark samples in Figure 4.17b were divided based on the borehole from which they were taken which shows some clustering despite all being metagranites. The variation of test results with depth was examined, but there was no clear relationship indicated by the limited data. The clustering suggests that even though the samples are all meta-granites, natural variability at the same project site influences the strength of the material. Further study is required to explore how natural variability within a specific rock type influences the relationships examined in the paper.

Despite the variability of the test results gathered, when a histogram of the ratio between DTS and BTS is examined (Figure 4.18) there is a clear indication that the DTS is over estimated by the Brazilian tensile test method. The over estimation may be small for igneous rocks, which exhibit a strong peak. However, the peaks for the metamorphic and sedimentary rocks are less well pronounced, leading to a greater over estimation. The results for each rock type are normally distributed. Based on the normal fit, the mean values of the DTS / BTS ratios are 0.86, 0.82, and 0.70 for the metamorphic, igneous and sedimentary rocks, respectively.

The igneous results show the smallest standard deviation and coefficient of variation, as expected since there tends to be more consistent grain size and stiffness in the igneous rocks.
tested and reported in the literature. Despite the variability of the data from the other rock types, there is an indication that the relationship between DTS and BTS is rock type dependent. Using the above ratios, an approximate estimate of the DTS can be estimated from BTS test results.

4.6 Discussion

The standardization of tensile strength testing for rock mechanics suggests both direct and indirect methods can be used. In light of the difficulties with direct tensile methods, indirect methods are more often conducted.

![Figure 4.18: Histogram of the ratio DTS and BTS for the main rock types (meta. = metamorphic, sed. = sedimentary, ig. = igneous, St. Dev. = standard deviation, and COV = coefficient of variation).](image)
The authors would go a step further and suggest that indirect samples be cut from the end of UCS samples whenever possible to get the best possible correlation between tensile and compressive damage thresholds. The samples should be examined carefully in order to ensure that they are of a similar nature and that the fabric if present is aligned properly for comparison.

Many rocks have a fabric, either metamorphic or depositional in origin, which influences the strength; this influence depends on the loading direction with respect to the orientation of the fabric. The weakest orientation is typically found between 30 and 60 degrees inclination and a recent study, by Dan et al. (2013), demonstrates similar results for Brazilian tensile strength.

The optimum comparison of tests from different samples with a fabric is one in which the failure plane interacts with the fabric in a similar manner, i.e. a vertical failure plane cuts across or parallel to the fabric. In the data collected from the literature, this information was not always given, and may account for much of the scatter in comparing DTS and CI with BTS results.

Observational evidence by other researchers (Griffith 1921, Tapponier and Brace 1976, Stacey 1981, Myer et al. 1992, Lee and Haimson 1993) suggests that the initiation of damage is via an extensional mechanism. In a DTS test, slip dislocation is not prevalent for most non-metallic solids (Lockner 1993). If slip dislocation is not able to occur, in compression of hard brittle rocks, then shear rupture must also be initiated by tensile failure at the micro scale. Since CI is a measure of the stress required to initiate cracking in a compression test, the theory supports the relationship found in this paper between the tensile strength and CI. The CI threshold has been found to be relatively insensitive to confinement in comparison to the CD threshold (Diederichs 1999, Martin 1994, Brace et al. 1966, Pestman and van Munster 1996).

Since the CD threshold is confinement dependent, crack growth during a Brazilian Tensile test could be suppressed in the outer 20-25 % of the radial dimension of the disk when the
loading width (2a ) is less than 16 (see Figure 4.3a). However; according to the ISRM (1978) suggested method for the Brazilian test, the difference between the load at primary fracture and the ultimate load capacity of the sample after fracture is roughly 5%. This would suggest that suppressed crack growth is not the leading factor causing BTS to be larger than DTS, but perhaps it does play a part in combination with other factors. The physical mechanism of failure is resisted by a small frictional force between the sample and the loading platen. Therefore the sample is unable to fail in a purely tensile manner (i.e. similar to a direct test). Markides et al. (2011) indicate that the frictional resistance only influences a narrow region near the platen. Another aspect which could influence the variation in tensile strength is the mineralogy. The rocks which are composed of dominantly one type of mineral appear to plot closest to the $DTS = BTS$ line and those which are likely to have variation in mineralogy have more scatter. For those samples which display large variations in mineralogy it would also be more difficult to select similar samples even within 10 cm of each other. More minerals contained within a sample with different moduli could influence crack propagation by forcing more tortuous fracture pathways. This would influence the BTS more, as the fractures must propagate through zones of confinement due to differences in grain stiffness and strength. Despite the influences discussed above there is a clear indication that the relationship between DTS and other methods of determining the tensile strength is rock type dependent.

4.7 Conclusions
The test results in the literature for tensile strength suggest that it is difficult to estimate the true tensile strength accurately from other laboratory results. An estimate from UCS values gives the most erroneous results. $UCS / T$ does give a good approximation for the Hoek and Brown (1997) constant $m$, however, this relationship should be used with caution to estimate tensile strength.
particularly if the $m$ value has not been determined from laboratory testing. Using CI to predict tensile strength generates less variability than using UCS and is in close agreement with $T = \text{CI} / 8$. It could be used for preliminary assessment in engineering projects. Actual testing is always recommended.

It was shown that the relationship with CI is also rock type dependent, similar to the relationship between DTS and BTS. In fact better correlation coefficients exist when using the main rock types, such that $f'\text{in DTS} = f'\text{BTS}$ ranges from approximately 0.9 for metamorphic, 0.8 for igneous and 0.7 for sedimentary rocks, for practical applications. Reducing the BTS values based on the main rock types improves the correlation with CI.

There is a large degree of scatter in the relationships presented in this paper, which is in part due to the differences in rock type and mineralogy. Closer examination of granite and metagranite data indicates that some of this scatter is also related to natural variability. This variability was examined based on the geographic location from which the samples came from and locally based on different boreholes from the same site. In both cases the scatter is reduced when the specific geographic information is used to further sub-divide the specific rock type (granite or metagranite examples in Figure 4.17).

Further study is required on igneous and sedimentary rocks to compare DTS with BTS and tensile strength with CI to confirm the findings of this review of the tensile strength of rock.

4.8 Acknowledgments
The authors would like to thank the Nuclear Waste Management Organization (NWMO) of Canada and the Natural Sciences and Engineering Research Council (NSERC) of Canada for funding this review. Special thanks is due to Dr. Evert Hoek for use of testing data and for discussions related to this paper.
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PART II: Excavation Damage Zone Characteristics in Mudrocks and Carbonates
Chapter 5: Observations from the Niagara Tunnel Project in the

Queenston Shale

5.1 Abstract

The Niagara Tunnel Project is a water diversion tunnel recently completed in Niagara Falls, Ontario, Canada. The tunnel was excavated using a 14.4 m diameter tunnel boring machine and passed through eleven formations. The rock types include limestone, sandstone, siltstone, shale and mudstone. Overbreak behaviour was divided into four zones, as observed by the author. The deepest overbreak values, 3-4 m typically, were located in the high horizontal stress areas which the tunnel was excavated through. Induced fractures propagate along the typically horizontal bedding fabric allowing the stress induced failure to be wide spread by the time it reaches the back of the cutter head, creating a large volume of overbreak. This was overcome by utilizing spiles and adjusting the tunnel alignment to minimize the duration of excavation in the Queenston Formation. The tunnel has been in operation since March 2013.

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3 This chapter appears as submitted to an international journal with the following citation:
5.2 Introduction

The Niagara Tunnel Project is a water diversion tunnel which was excavated by a 14.4 m diameter Tunnel Boring Machine (TBM). The tunnel passes through the entire horizontal stratigraphy of the Niagara Escarpment, from the cap rock of the Lockport Formation down into the upper Queenston Formation. It passes under the buried St. Davids Gorge, which is in-filled with glacial material, to make its connection with the existing power canal system above the hydroelectric Sir Adam Beck Generating Stations (SAB GS). The project is now in full operation. The project enables Canada to utilize more of its hydroelectric potential from the Niagara River and help the owner, Ontario Power Generation (OPG), meet the growing demand for green energy.

The Niagara River flows from Lake Erie to Lake Ontario, with an average volume of 6000 m$^3$/s (Delmar et al. 2006). A 1950’s treaty between Canada and the United States of America stipulates the volume of water flowing over Niagara Falls must be maintained at 2832 m$^3$/s during the daytime from April to October and at 1416 m$^3$/s at other times (Delmar et al. 2006). The remaining volume of water is shared equally between the two countries and when one is not utilizing the full capacity, the other can utilize the available power generating capacity (Delmar et al. 2006).

Hydropower generation on the Niagara River, on the Canadian side, is generated at the SAB GS and consists of SAB 1 and SAB 2, operating since 1922 and 1954, respectively. Prior to completion of the new tunnel, the Queenston-Chippawa Power Canal and twin 13.7 m internal diameter tunnels diverted water to the SAB GS from above Niagara Falls. The twin tunnels surface prior to crossing St. Davids Gorge, into a joint canal system for the various feeders of the SAB GS, including a pumped storage reservoir, as shown in Figure 5.1.
Figure 5.1: The Sir Adam Beck Generating Station (SAB1 and SAB2) water diversion system; including the Queenston-Chippawa Power Canal, twin existing tunnels (Tunnel), the Pumped storage reservoir and the new Niagara Tunnel.
Figure 5.2: Long section of the Niagara Tunnel Project showing major geological groups and the old tunnel alignment. Overbreak zones were encountered within the tunnel excavation in: Zone 1) formations above the Queenston, Zone 2) Whirlpool – Queenston contact, Zone 3) St. Davids Buried Gorge, and in Zone 4) due to the regional stress field (Perras and Diederichs 2009).

With increasing demand for power in Canada, OPG began constructing the Niagara Tunnel under the Canadian city of Niagara Falls, Ontario, in 2006. The new tunnel has increased diversion capacity of the SAB GS to 2300 m$^3$/s from 1800 m$^3$/s. This increase will mean that available water for diversion to Canada will only exceed the SAB capacity about 15 % of the time, rather than the previous 65 % (Delmar et al. 2006).

This chapter is intended to document the geological and geotechnical observations within the first 3.1 km of the Niagara Tunnel, which were observed by the authors. This portion of the tunnel intersected all the formations that would be encountered in entire tunnel alignment (Figure 5.2) and includes the most challenging aspects of the rock mass behaviour.
5.3 Geological History

The tunnel passes through eleven formations of the Appalachian sedimentary basin (Figure 5.2). The Appalachian basin, which covers most of southern Ontario, consists of Paleozoic strata deposited over Precambrian basement rocks. The basin is bounded by the Algonquin and Findley arches in the West, the Frontenac arch in the North and the Appalachian Mountain Range (Appalachian Front) in the East and South (Figure 5.3), from where the sediments were eroded during the Ordovician and Silurian periods. The arches are Precambrian granitic rock highs.

Figure 5.3: Regional geological setting showing the boundaries of the Appalachian sedimentary basin and depth contours in meters above sea level (modified from Mazurek 2004).
Figure 5.4: Stratigraphic section (Perras and Diederichs 2007) of the formations which outcrop along the Niagara Escarpment near Niagara Falls, Ontario, Canada. Thicknesses are those observed in the Niagara Tunnel Project or as indicated (*) by Haimson (1983).

<table>
<thead>
<tr>
<th>Age</th>
<th>Group</th>
<th>Formation</th>
<th>Description</th>
<th>Thickness (m)</th>
<th>Symbol</th>
</tr>
</thead>
<tbody>
<tr>
<td>Middle Silurian</td>
<td>Albemarle</td>
<td>Lockport</td>
<td>Grey crystalline dolomitic limestone</td>
<td>16.8 - 20.3*</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Decew</td>
<td>Dolomite &amp; grey mudstone</td>
<td>2.1 - 4.0*</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Clinton</td>
<td>Rochester</td>
<td>Dark grey calcareous shale &amp; interbeds dolomite</td>
<td>18.7</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Irondequoit</td>
<td>Grey to reddish limestone</td>
<td>3.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Reynales</td>
<td>Light grey dolomite</td>
<td>3.3</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Neahga</td>
<td>Green shale</td>
<td>2.1</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Thorold</td>
<td>White sandstone</td>
<td>2.7</td>
<td></td>
</tr>
<tr>
<td>Lower Silurian</td>
<td>Cataract</td>
<td>Grimsby</td>
<td>Sandstone &amp; red shale beds</td>
<td>13.3</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Power Glen</td>
<td>Grey shale &amp; white sandstone</td>
<td>9.7</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Whirlpool</td>
<td>Light grey sandstone</td>
<td>6.0</td>
<td></td>
</tr>
<tr>
<td>Ordovician</td>
<td>Queenston</td>
<td></td>
<td>Red siltstone and argillaceous limestone</td>
<td>335*</td>
<td></td>
</tr>
</tbody>
</table>
The formations within the Appalachian basin lie relatively flat, dipping 6m/km (Yuen et al. 1992) with local variations. Topographically, Southern Ontario is relatively flat, with the exception of the Niagara Escarpment, a cuesta running from Lake Huron at the Bruce Peninsula to Rochester, New York. The escarpment has been cut into by river erosion at numerous locations and most notably by the Niagara River, which has created the world renowned Niagara Falls. The river systems tend to follow the regional joint systems. Although the geology may appear relatively straight forward, key challenges had to be overcome on the Niagara Tunnel Project.

5.3.1 Stratigraphy
The eleven formations which the tunnel transects outcrop along the Niagara Escarpment in the Niagara Region. The stratigraphic column is shown in Figure 5.4 along with observed thicknesses of each formation as crossed along the tunnel, where applicable.

During Ordovician time, the east coast of North America was a much different place than it is today. Volcanic rocks in the northern part of the Appalachians indicate that subduction was occurring and island arcs were being formed. Away from the coastal side of the Appalachians the sediment supply, from the mountain building, increased and became more argillaceous westward and covered the older shallow-shelf limestone. The sediment volume increase created sandstone units close to the edge of the mountains and formed shale units further away. The shale units are predominately the Blue Mountain, Georgian Bay and Queenston Formations (Mazurek 2004).

The Queenston Formation, which gradationally overlies the Georgian Bay Formation, is Upper Ordovician in age and is part of a compound deltaic, shallowing upward sequence (Stearn et al. 1979) of a semiarid region (Brogly et al. 1998). The Formation is a red and grey argillaceous mudstone with occasional siltstone and sandstone interbeds and fossiliferous siltstone to limestone beds (not observed in the tunnel) near the base (Rigbey et al. 1992 and
Brogly et al. (1998). In the project area it is over 300 m thick. The sediment was provided by rivers flowing down from the barren Appalachian highlands, mountains built during the Appalachian Orogen, and are mostly composed of red mudstone with a green-grey mottled colouring in places, that developed west of the highlands toward the Niagara Region during the close of the Ordovician period (Stearn et al. 1979). The mudstone was deposited in a lagoonal environment, which was periodically inundated by sea water (Stearn et al. 1979). This inundation, into areas of restricted flow, allowed for the concentration of evaporite minerals in the shallow waters. This has resulted in ground water which is highly connate today (Rigbey and Hughes 2007) and gypsum dispersed throughout the formation. Brogly et al. (1998) reported that it occurs as nodules or thin laminae and these were observed in the Niagara Tunnel. A sharp increase in nodules, gypsum/calcite filled burrows and laminae were observed below elevation 55 m.

The greenish-grey banding in the Queenston Formation is formed as a secondary process called reduction. The mottled green-grey colour is common in the tunnel and exists as horizontal beds, irregular spots and along some vertical joints (Figure 5.5a). The process is simply a chemical reaction, whereby the Fe(III), or ferric iron, of the red Queenston is reduced to Fe(II), or ferrous iron, of the greenish-grey bands by ground water flow.

At the close of the Ordovician and prior to the start of the Silurian, an erosional period occurred where the upper-most part of the Queenston Formation was removed (Mazurek 2004) prior to the deposition of the Whirlpool Formation. This represents a significant unconformity between the Queenston and Whirlpool formations.
Figure 5.5: Geological observations from the Niagara Tunnel Project; a) Reduction banding (greenish grey bands) within the Queenston Formation. Note that the sub-vertical to vertical joints were only observed in the upper most ~50 m of the Queenston. b) Complex interbeds of shale within the Thorold sandstone, c) two continuous thin shale beds, less than 0.020 m thick, in the upper Reynales Formation, and d) a bioherm (Irondequoit limestone) protruding up into the Rochester Formation (Perras 2009).
The Cataract Group is Lower Silurian in age and was deposited in a deltaic and shallow marine environment (Currice and Mackasey 1978). It includes the Whirlpool, Power Glen and Grimsby Formations. The Whirlpool sandstone, which disconformably overlies the Queenston (Mazurek 2004), represents a transgressive state of a sea that washed and rounded quartz sand grains (Stearn et al. 1979). The Whirlpool Formation has been observed to be on average 6 m thick in the Niagara Tunnel.

The Power Glen Formation is a grey shale with white calcareous sandstone interbeds. It can be broken down into two units; the upper unit being a light grey sandstone with grey interbeds of shale, and the lower unit being predominately grey shale with interbeds of light grey sandstone. The formation was observed to be roughly 10 m thick in the tunnel, with sub-units of equal thickness. The deposition of this shale rich formation marks an increase in sea levels. Winder and Sandford (1972) state that minor fluctuations in the sea level allowed the depositional material to alternate between sand, clay and calcareous shell fragments, forming a shale sequence with interbedding of sandstone and limestone. The latter was not observed in the tunnel.

The Grimsby Formation consists of thin beds of red shale, 0.025 to 0.200 m thick, interbedded with red fine grained sandstone. Ripple marks were observed in hand samples suggesting deposition in a sub-aqueous environment. On the other hand, mud cracking has also been observed, suggesting alternating flooding and exposure. Similar to the Power Glen, minor fluctuations in the sea level provided alternating depositional materials creating laterally continuous shale beds in a predominantly sand rich depositional environment. The Grimsby Formation was observed to be on average 13.3 m thick.

The Clinton Group is Middle Silurian in age and comprised of the Thorold, Neahga, Reynales, Irondequoit and Rochester Formations, which were deposited in a shelf edge
environment (Winder and Sanford 1972). Reworked Grimsby detritus formed the Thorold Sandstone as sea levels increased. Tesmer (1981) also suggests that portions of the Thorold are windblown beach sands because of frosted, round quartz grains. Flat and undulating interbedded shale bands can be observed in the tunnel walls of the Niagara Tunnel Project. This was common in the sandstones observed and was most pronounced in the Thorold Formation which is shown in Figure 5.5b. It is a relatively thin unit, on average 2.7 m thick, as observed in the tunnel.

With continuing sea level changes in an intertidal and lagoonal environment, the Neahga and Reynales Formations were deposited (Winder and Sanford 1972). The Neahga Formation is a thin, 2.1 m thick, fissile dark green shale layer which marks the transition from siliciclastic to the calcareous rich Reynales Formation. The Reynales Formation is a light grey crystalline dolomite which was on average 3.3 m thick in the Niagara Tunnel. The Reynales contains two continuous shale beds in the upper 0.6 m of the formation (Figure 5.5c). These beds can be used as marker layers for mapping the formation as they were also observed in the previous hydro tunnels constructed in the 1950’s (HEPC 1953).

The Irondequoit Formation is a crinoidal dolostone, with occasional bioherms, which can be seen in Figure 5.5d to be draped by the dolomitic muds forming the Rochester Formation. The Irondequoit is a light grey to pinkish dolomitic limestone which was observed to have an average thickness of 3.0 m in the Niagara Tunnel.

The Rochester Formation is a dark grey dolomitic shale (Winder and Sanford 1972), which was observed to be on average 18.7 m thick in the Niagara Tunnel. It is very finely bedded and contains occasional limestone interbeds.

Patch reefs, forming the Decew and the lower Lockport Formation and regional reefs, forming the upper Lockport and Guelph Formations (Tesmer 1981), completely surrounded the
Michigan Basin and allowed for the precipitation of evaporites of the Salina Formation (Sanford 1968), which overlie the Lockport. The Salina Formation is not present in the Niagara Tunnel Project area and the erosion resistant Lockport Formation forms the cap rock of the topographic highs in the Niagara Region.

5.3.2 Topography and Quaternary Geomorphology

There are three main topographic features in the Niagara Region. They are St. Davids Gorge, the Niagara Escarpment and the Niagara River Gorge. Each feature has in part or in full been shaped by glacial activity. St. Davids Gorge was in existence prior to mid – Wisconsin time (Hobson and Terasmae 1969) and the Niagara Escarpment shows evidence of existence prior to the last ice advance in Wisconsin time (Hugh 1958). According to Pengelly (1997) the Niagara River Gorge is post-Wisconsin, but the water carving the gorge would be of glacial origin, draining from further north via the now Great Lakes.

5.3.2.1 St. Davids Buried Gorge

Spencer’s work from 1907 confirmed observations by Lyell from 1845 that a buried gorge existed between the current Whirlpool and the village of St. Davids in the Niagara Region. Such investigations have retrieved pollen and wood samples, which confine the timing of the deposition of the infilling detritus between Late and Middle Wisconsin (Abidi et al. 1992). The sediments are lacustrine, glaciofluvial and glacial in origin (McKenzie 1990). Karrow and Terasmae (1970) indicate that a low energy environment would be necessary for the deposition of pollen bearing sediments and suggest that St. Davids Gorge was dammed by glacial ice. Detailed sediment descriptions by Abidi et al. (1992) confirm that glacial advances occurred many times during the filling of the gorge as marked by the interbedded glacial till. Understanding the timing of the erosion of St. Davids Gorge is somewhat more difficult. Karrow and Terasmae (1970)
report that erosion could have occurred during the Port Talbot intertidal of the Early Wisconsin (~65 000 to 79 000 BP), or the interglacial Sangamonian period (~122 000 to 132 000 BP), prior to the advance of the first Wisconsin ice.

A number of theories regarding the reason for the location of St. Davids Gorge have been postulated, although none have been published in detail. Spencer (1907) indicates that the limestone crest of the gorge in the Whirlpool area has been rounded and suggests that a pre-existing valley was scoured by glacial activity and subsequently eroded by melt water. Others have suggested that it may be a fault or a tunnel valley, a valley carved below glacial ice. Investigations and observations from the Niagara Tunnel Project have helped to gain a better understanding of the possible origin of St. Davids Gorge and further discussion follows later in this chapter.

5.3.2.2 Niagara Escarpment
The Niagara Escarpment is a cuesta-type landform, running roughly north-south which extends from Northern New York State, along the western shore of Lake Ontario and up to the tip of the Bruce Peninsula, where it is submerged under Lake Huron. The escarpment is capped with the resistant limestone of the Lockport-Guelph Formations in the Niagara Region or the Amabel Formation further north. Barlow’s (2002) study of the escarpment indicates that creep deformation in under lying shale layers (Power Glen and Queenston), due to stress levels close to the shale strength, creates zones of tension in the more competent layers above (Lockport, Irondequoit, and Reynales). The zones of tension create vertical fractures, which then can be widened through karst or freeze thaw processes. The present day deformation process, suggested by Barlow (2002), of the Niagara Escarpment may be different than the historical.
The origins of the Niagara Escarpment are still not clearly defined in the literature. Barlow (2002 and 1995, respectively) suggested two possibilities - one is river erosion parallel to the face, which was subsequently disturbed by glacial activity and the other is a wave cut cliff in an early glacial lake. In either case the stratigraphy and the resistance of the upper Lockport limestone, has played an important part in preserving this topographic feature.

5.3.2.3 Niagara River Gorge
The erosion of the Niagara River Gorge was initiated at the end of the last glacial period as part of a multi-outlet glacial lake system (Calkin and Brett 1978). Pengelly et al. (1997) described two main glacial lakes, Lake Tonawanda to the east of the existing river, and Lake Wainflekt to the west. These ancient lakes had multiple outlets flowing over the existing Niagara Escarpment and as lake levels fluctuated, the flow was directed to Queenston, Canada where the Niagara River Gorge began forming at the edge of the Niagara Escarpment.

The erosion of the Niagara River gorge to its present location at Niagara Falls is reported, by Pengelly (1997), to have taken 10 000 to 12 000 years. The total length of the gorge is 11 km and the present erosional rate at the Horseshoe Falls is roughly 0.3 m per year. This erosional rate is much reduced from the past, due to water diversion for hydropower.

These three topographic features bound a block of rock in the launch portal area of the tunnel and the tunnel follows the Niagara River Gorge once it passes below St. Davids Gorge. Numerical studies by Perras and Diederichs (2007) indicate that influence of gorge erosion can be observed up to 1 km from the edge of the gorge. The gorge erosion can modify the stress field and even damage the rock, reducing the strength. This in turn can influence the excavation performance of underground projects in the vicinity of such topographic features.
5.4 Engineering Properties

Over the course of the planning period for the Niagara Tunnel Project site investigation studies were carried out to determine many engineering properties; including structural styles, rock properties (intact and rock mass), in-situ stresses, and swelling. The discussion in this paper focuses on unconfined compressive strength (UCS), stress and structure, as the swelling behaviour has been well documented by Rigbey and Hughes (2007).

5.4.1 Structural Features

Sanford et al. (1985) described the Niagara megablock which is bounded by three major physiographic features, namely the Niagara Escarpment, the Niagara River Gorge and St. Davids Gorge. These three features intersect to form a large triangular block of rock, of which the Queenston Formation forms the base.

Regionally, a weathered fracture network near the bedrock surface is pervasive throughout the Niagara Region (Novakowski and Lapcevic 1988). Bedding plane fractures are predominant in the Niagara Region, with vertical jointing being widely spaced (Novakowski and Lapcevic 1988). Superimposed on the regional structure and associated with the three physical features are considerable stress relief fractures along the gorges and the escarpment, due to the tensile zone created by the adjustment in the horizontal stresses around these topographic features. Bedding plane joints without shear indicators were not observed in any of the formations intersected by the tunnel, except in the Queenston Formation.

Widely spaced horizontal shears have been observed in the upper elevations of the Queenston Formation in a test adit driven for the Niagara Tunnel Project. These sheared bedding planes have a spacing of 8 – 10 m and are sometimes filled with silt gouge (Rigbey et al. 1992). The gouge filled shears are close to the Whirlpool contact and decrease with depth (Rigbey et al.
1992). Similar features were observed in the tunnel in the area north of the St. Davids Gorge and are believed to be the result of stress relief from gorge erosion. These were observed up to 700 m away from the gorge centre. Possibly these extend beyond, but were not observed in the tunnel. In addition to the sheared bedding planes, slickensided shears are described as a ubiquitous features (Russell and Harman 1985) that have been observed in both weathered and core samples of the Queenston and have been described as settlement features during deposition and consolidation.

Other lineaments in the Niagara region are reported as inclined shear planes in the Rochester and Neagha formations, of which several have been reported (Mazurek 2004). These features may be formed due to direct loading – unloading by an ice sheet (Karrow and White 2002) or drag forces, since there is little evidence of tectonic influences after Middle Silurian time in the Niagara Region (Stearn 1979). A report by Novakowski and Lapcevic (1988) suggests another lineament, at the base of the Rochester Formation, as determined by isopach mapping. Sanford et al. (1985) reports a fault network, although this has not been observed in the field. However, an inclined shear plane with slickensides in the Neagha Formation was only observed at one location in the tunnel.

Four sub-vertical to vertical joint sets and one horizontal set, parallel to bedding, were identified within the Niagara Region prior to construction of the diversion tunnel. The strike of the sub-vertical sets are 005°, 045°, 085° and 135° (Perras 2009). The joint sets are roughly parallel to the major topographic features of the region, namely the Niagara Escarpment, St. Davids Buried Gorge and the Niagara River Gorge. Regionally the joints are typically planar, slightly rough, and can contain gypsum or calcite infilling. All of the surface joint features in the Niagara Region are widely to very widely spaced. Observations from the tunnel are in agreement with the regional data and will be discussed in more detail later.
5.4.2 Rock Properties

Testing of rock samples from boreholes and from within a test adit was conducted to determine the engineering properties of the formations to be encountered by the tunnel. Testing provided the Hoek-Brown material constant ($m_i$) (Hoek et al. 2002), compressive strength (UCS), Young’s Modulus ($E_i$), and posson’s ratio ($\nu$).

Typical values are shown in Table 5.1 for samples tested perpendicular to bedding. Rock Quality Designation (RQD), Rock Mass Rating (RMR), Geological Strength Index (GSI) and Swell potential were also determined for the various formations. The RMR and GSI values are also reported in Table 5.1.

Table 5.1: Typical rock mass and intact properties derived from investigations for the Niagara Tunnel Project (Perras 2009).

<table>
<thead>
<tr>
<th>Formation</th>
<th>RMR/GSI</th>
<th>$m_i$</th>
<th>UCS (MPa)</th>
<th>$E_i$ (GPa)</th>
<th>$\nu$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lockport</td>
<td>70/80</td>
<td>9</td>
<td>151</td>
<td>67</td>
<td>0.37</td>
</tr>
<tr>
<td>Decew</td>
<td>70/70</td>
<td>9</td>
<td>128</td>
<td>51</td>
<td>0.3</td>
</tr>
<tr>
<td>Rochester</td>
<td>64/77</td>
<td>7</td>
<td>41</td>
<td>11</td>
<td>0.3</td>
</tr>
<tr>
<td>Irondequoit</td>
<td>72/82</td>
<td>10</td>
<td>89</td>
<td>60</td>
<td>0.32</td>
</tr>
<tr>
<td>Reynales</td>
<td>67/77</td>
<td>9</td>
<td>101</td>
<td>33</td>
<td>0.25</td>
</tr>
<tr>
<td>Neaha</td>
<td>56/66</td>
<td>6</td>
<td>18</td>
<td>4</td>
<td>0.45</td>
</tr>
<tr>
<td>Thorold</td>
<td>78/83</td>
<td>18</td>
<td>129</td>
<td>53</td>
<td>0.22</td>
</tr>
<tr>
<td>Grimsby Shale</td>
<td>60/60</td>
<td>7</td>
<td>35</td>
<td>8</td>
<td>0.25</td>
</tr>
<tr>
<td>Grimsby Sandstone</td>
<td>70/75</td>
<td>17</td>
<td>146</td>
<td>43</td>
<td>0.24</td>
</tr>
<tr>
<td>Power Glen Shale</td>
<td>60/60</td>
<td>7</td>
<td>35</td>
<td>8</td>
<td>0.25</td>
</tr>
<tr>
<td>Power Glen Sandstone</td>
<td>63/68</td>
<td>17</td>
<td>152</td>
<td>59</td>
<td>0.21</td>
</tr>
<tr>
<td>Whirlpool</td>
<td>85/87</td>
<td>19</td>
<td>180</td>
<td>50</td>
<td>0.19</td>
</tr>
<tr>
<td>Queenston</td>
<td>65/65</td>
<td>8</td>
<td>46</td>
<td>16</td>
<td>0.35</td>
</tr>
</tbody>
</table>
Most of the formations in the Niagara area have thin interbedded layers, which in most cases can be considered to be part of the intact rock. However; the Grimsby and Power Glen formations have thick distinct shale and sandstone beds. In this case the strength properties were separated into shale and sandstone (Table 5.1). In the more massive formations, like the Queenston, the strength can still vary considerably. As can be seen in Figure 5.6, the strength variation for the Queenston falls within a 40 MPa range above elevation -20 m.

Below elevation -20 m the strength range increases markedly. The cross section at the bottom of Figure 5.6 shows that the strength increase occurs within the rocks found approximately 25 m below the deepest area of St. Davids Gorge near the tunnel alignment. The strength data comes from borehole SD-3. This strength increase can be in part due to changes in the composition (e.g. variations in silt or clay sized material) or in relation to the physiographic features which may cause local strength reduction where the bedrock has been able to relax in the vicinity of the gorges.

5.4.3 Stress Field
High horizontal stresses exist in the sedimentary rocks of Southern Ontario. These stresses are the result of tectonic activity during the Appalachian mountain building, from sedimentary basin effects and from glacial loading and erosion. The stress ratio (Ko) is reported to be in the order of 3 – 5 (Perras and Diederichs 2007), with magnitudes ranging between 9 – 24 MPa in the Queenston Formation (Yuen et al. 1992). Stresses for the Niagara Tunnel Project were measured using down borehole hydro-fracturing methods in most cases, with the exception of overcoring methods which were completed from within a test adit in the Queenston Formation.
Figure 5.6: The relationship between depth and unconfined compressive strength data from measurements on the Queenston Formation (modified from Perras 2009) and below the bedrock elevation along St. Davids Gorge (bedrock depths from Abidi 1992). The high strength values come from borehole SD-3 and the strength increases approximately 25 m below the deepest area of St. Davids Gorge near the tunnel alignment.
There can also be a sharp difference in the state of stress from formation to formation due to elastic properties (Haimson 1983). Stress values, as measured during the investigation stage of the project, are shown in Figure 5.7. Below an elevation of 40 m there is a sharp increase in the magnitude of both the major and minor horizontal stresses. This elevation corresponds roughly to the deepest bedrock surface elevations along the Niagara River Gorge, as shown in the developed section in Figure 5.7. This stress magnitude transition results in an increase of up to 12 MPa and likely deviates due to the irregular nature of the gorge bottoms (Niagara and St. Davids). The deepest part of the tunnel (45 m) passes close to the stress magnitude transition, which likely contributes to variations in the observed overbreak dimensions. This will be discussed further in the observations section (5.6) of this paper.

5.5 Tunnel Design
The primary concerns for design of the Niagara Tunnel were swelling ground and in-situ stress conditions. To deal with the swelling issues it was decided to use a two pass tunneling system where the primary rock support is installed from the TBM and on the second pass an impermeable membrane and final concrete liner are installed.

5.5.1 Tunnel Support
The primary rock support was installed in two areas, the L1 area, directly behind the cutterhead and the finger shield, roughly 6 m from the face, as well as at the L2 area, 35 m back from the face (Figure 5.8). The primary rock support consisted of wire mesh, C-channel and rock bolts, although I-beam rings were used early on.
Figure 5.7: In-situ stress measurements (maximum and minimum horizontal) taken along the tunnel alignment showing variation with depth (modified from Perras 2009) and developed section along the Niagara River (A-A’) and St. Davids Buried Gorge (B-B’) showing the depth to bedrock (Philbrick 1970 and Abidi 1992) or river bottom (NOAA 2004), where data was available.
Figure 5.8: The TBM and trailing gear in the launch cut at the portal to the Niagara Tunnel (Perras 2009).
The wire mesh was held in place behind the C-channel which was in turn secured using swellex rock bolts. The C-channel is a steel arch piece (half of an I beam) formed to the curvature of the tunnel circumference with holes through which the rock bolts can be installed. Swellex rock bolts are a double-folded steel tube which is expanded into the borehole using water pressure. The expansion generates contact friction resulting in a fully bonded bolt. The spacing between successive C-channels was adjusted according to the ground conditions and additional rockbolts could be installed in between C-channels, if necessary. The C-channels typically would span a circumferential distance of 6.5 m, ending at the base of the haunch area (the haunch being defined as between 1 and 2 or 10 and 11 o’clock). Since the rock was scaled before the installation of rock support in the L1 area, the C-channels were fitted to the overbreak profile, if present. Crown shotcrete could also be installed in the L1 area when required; however, the majority of the shotcreting was done in the L2 area. Here a full ring of shotcrete could be sprayed to cover the steel support. Additional rock bolts could be installed, if necessary, before shotcrete application.

On the second pass an impermeable membrane is installed to prevent fresh water from the tunnel coming in contact with the rock. The membrane is bedded against a relatively smooth layer of shotcrete and sandwiched between the final concrete liner, as shown in Figure 5.9. Fresh water contact could lead to swelling of many of the shale and mudstone formations, of which the Queenston mudstone has the highest swelling potential, as discussed by Rigbey and Hughes (2007).

Grouting of the final concrete liner is the last work to be conducted. Grouting applies a pre-stress to the concrete in excess of the internal water pressure. Prestressing avoids the need for installation of reinforcement in the concrete liner.
Figure 5.9: Drawing of tunnel lining strategy to minimize and eliminate swelling in the shale layers for the Niagara Tunnel Project by adding a membrane (from Rigbey and Hughes 2007).

5.5.2 The TBM

The TBM for the Niagara Tunnel Project was an open gripper, main beam machine with a diameter of 14.44 m. At the time of construction it was the largest hard rock TBM of its kind in the world. Harding (2007) describes the TBM to be approximately 45 m long with an attached 105 m long trailing gear system, which houses all the supporting components for the operation of the unit. There were 85 disk cutters on the head, each with a diameter of 0.508 m. The cutter
disks could be back loaded into the cutterhead to save time during replacement. The cutterhead is powered by 15 electrical motors for a total of 6330 HP and 18 800 kNm torque at low speeds of 2.4 rpm. The recommended cutterhead thrust pressure was 28 MPa and for the grippers 21 MPa.

The cutterhead (stiff shield) was 4.1 m long and an additional ~2 m flexible finger shield extended from the back of the cutterhead. There were two rock drills located in L1, which were used to drill holes for rock bolt installation. The drills were mounted on a gear ring, centered about the main beam, which could travel forward and backward. The drills themselves could also rotate in the drilling plane to allow for multiple drilling angles. There was also a forward probe drill attached to a circular traveler allowing for movement both axially and radially. The forward probe drill was limited to a 30 degree inclination above the cutterhead and was used for advance probing, for pipe spile installation (Figure 5.10), and for scaling in the crown area above the cutterhead. A scissor lift brought C-channel and ring pieces forward for assembly and installation. Various working platforms were modified and replaced with two-man lifts installed to increase the mobility of the support crews.

The secondary support was installed almost 45 m from the working face, in the L2 area. Rock bolts, mesh and shotcrete could be applied at this location by track mounted equipment. The tracks allow for both radial and axial movement similar to the rock drills at L1. Two shotcrete robots applied enough shotcrete to cover the primary support elements to achieve a smooth surface for installation of the water proof membrane. The water proof membrane was then installed followed by the 0.6 m concrete liner, as part of the second pass system.
5.5.3 TBM Modifications

Modifications to the TBM were ongoing during tunnel excavation to fine tune the system. These modifications ranged from addition of a face foam system to modifying the rock support equipment. An overview of the modifications is discussed by Gschnitzer and Goliasch (2009). Those modifications which helped improve the rock support installation process are briefly discussed below as they influence the behaviour of the rock mass.
Figure 5.11: Highlighted modifications to the TBM at Niagara to decrease the volume of overbreak reaching the invert directly behind the cutter head as shown in a). The modifications include b) moving platforms and mesh erector removed and replaced with c) man baskets, and d) forward drill for spilling and scaling.

Significant overbreak resulted in a large volume of rock falling to the excavation invert behind the cutter head (Figure 5.11a). The modifications to the drilling equipment and the flexible finger shield improved the timing of rock support installation, bringing it forward almost 1 m closer to the excavation face, to help reduce the rock falling behind the cutter head. Early stair access to the crown and a mesh erector (Figure 5.11b) were removed to make room for man baskets (Figure 5.11c), which also improved the installation of rock support by improving worker
flexibility and access. These two modifications helped to enhance the performance of the TBM drive. A forward drill (Figure 5.11d) was also used extensively to help scale the overbreak above the cutter head. Other factors helped indirectly as well, such as dust control sprayers and invert floor conveyors for overbreak on the invert. Minimizing overbreak, which fell behind the cutterhead (Figure 5.11a) and damaged components of the TBM, was paramount in improving the advance rates.

5.6 Observations from the Tunnel

This paper deals with observations made in the first 3.1 km of the tunnel starting at the outlet portal located near the pump storage reservoir. The tunnel traversed the entire stratigraphic section on a 7.82% slope for 1.2 km and from this point began to rise on a gentle grade, 0.1%, beneath the buried St. Davids Gorge (Figure 5.12). Once past the gorge, the tunnel alignment was modified to reduce the amount of excavation in the Queenston, due to difficult tunneling conditions. The TBM connected at the opposite end of the tunnel with a grout gallery, which was used to seal the final leg of the TBM drive under the Niagara River, before emerging into a coffer dammed area next to the shoreline. The observations were made for formation and rock descriptions, geological features, overbreak, structural features and stress induced features. The most significant observations are discussed below, with an emphasis on the impact to tunnel construction.

To facilitate the discussion, the tunnel observations are divided into 4 zones (Figure 5.2) based on the observed behaviour of the overbreak. Zone 1 includes the formations above the Queenston Formation, Zones 2 and 4 are border regimes with little or no impact from St. Davids Gorge and Zone 3 encompasses the area influenced by the gorge.
Figure 5.12: A block diagram illustrating the observed joint orientations with respect to the tunnel and St. Davids Buried Gorge. The arrows indicate the direction of stress relief.

5.6.1 Structural Features

Within Zone 1 little or no jointing was observed in the Lockport, Decew, Rochester, Irondequoit, and Thorold formations. Jointing was predominantly in the Reynales where vertical joints were common and of short persistence (<0.5 m trace length). The common strike orientations were 035° and 090° which are similar to the surface measurements. Joints occurred in the Grimbsy, Power Glen and Whirlpool formations as short (<0.3 m trace length), discontinuous planes but generally were very widely spaced. On one occasion a large wedge was supported in the Power Glen as the result of a vertical joint trending nearly parallel to the tunnel orientation. Jointing in
the Neaghe Formation, because of its fissile shale nature, was difficult to observe, with the exception of the inclined shear mentioned previously.

To better describe the relationship between the structural features observed in the tunnel in the Queenston Formation and St. Davids Gorge area, a block diagram was generated to illustrate the relationship (Figure 5.12). The observations are stated below and their relationship to the gorge is discussed.

In Zone 2, a joint set was observed in the tunnel near the upper elevation of the Queenston Formation, which is laterally continuous over more than 20 m, as it intersected the tunnel, wall to wall, at an angle of 45°, with an orientation of ~120° azimuth, or sub-parallel with St. Davids Gorge (see Figure 5.12, number 1). This joint is close to one of the regional joint sets. A weak conjugate set to the above set was also observed at an orientation of 177°. It should be noted that over a distance of approximately 280 m, starting near the Whirlpool–Queenston contact, only 13 of these joints were observed and can be considered to be very widely spaced. This continuous joint set is characteristically green from reduction either side of the joint for approximately 0.05 – 0.10 m and planar, rough, with calcite infilling ranging from <0.001 m to 0.010 m in thickness. This joint set was first observed at the Whirlpool – Queenston contact, where it terminated. It also disappeared at greater depths as the tunnel advanced towards St. Davids Gorge, suggesting that this set could be related to stress relief and resulted from a stiffness incompatibility between the Whirlpool and Queenston formations.

Stress relief was observed along sheared bedding planes (horizontal) and inclined shears (Figure 5.13), which showed 0.1 – 0.6 m of offset between beds. A bedding plane shear was first observed almost 700 m away from the apex of St. Davids Gorge and was similar in nature to the sheared bedding planes observed in the test adit excavated during the investigation phase of the
project. The horizontal and inclined structures were included in the influence area of the gorge, Zone 3, and are illustrated in Figure 5.12 as numbers 2 and 3 respectively.

In addition to the shear structures, a vertical joint set with an orientation of 080° was observed, similar to the regional trend (Figure 5.12, number 4). This joint set was tight, planar, and smooth with an average spacing of 0.3 - 1.4 m. It was only encountered locally in Zone 3 directly under the gorge.

![Figure 5.13: Sheared zone underneath St. Davids Buried Gorge as observed in the Niagara Tunnel. A green reduction band and a vertical joint are offset. The sheared zones have localized pockets of enlarged shearing as indicated in the photo and follow thinner sheared surfaces of longer extent (Perras 2009).](image)
Zone 4 had very few structural features of note, throughout the observed section. The overbreak was only locally influenced by structural features, other than bedding planes which were present in all formations. The controlling factor on the development of the overbreak in this zone was the stress field.

5.6.2 Overbreak

The general geometry and style of the overbreak from each of the 4 zones are shown in Figure 5.14 and it should be noted that the crown observations were made directly behind the cutter head and the flexible finger shield, approximately 5 m from the face (after TBM modifications) and that support was installed at the end of the flexible finger shield. The depths and widths of the overbreak in the crown, sidewall, and invert are summarized in Table 5.2 for each formation above the Queenston Formation in overbreak Zone 1 and in Table 5.3 for each overbreak Zone (Zones 2-4) within the Queenston Formation.

Overbreak observed in the tunnel was controlled by the high stress, bedding, the UCS of the intact rock and locally by jointing. Overbreak was encountered in the Rochester, Neagha, Grimsby, Power Glen and Queenston formations, with minor loosening and fracturing in other units along bedding planes. The formations where overbreak occurred had RMR values of less than 65 and UCS values of less than 50 MPa. With the high horizontal stress ratio, the instability is focused in the crown and invert. Sidewall overbreak was limited in the upper units above the Queenston and associated with vertical jointing. Minor side wall damaged occurred in the Queenston.

5.6.2.1 Zone 1 Overbreak

Zone 1 overbreak observations of significance were related to induced fracture growth at shale partings, thin bedding plane fall out in the crown, gravity slabbing, local wedge failure, minor
stress induced slabbing, and invert loosening. The typical overbreak style can be described as bed parallel fracturing, with fall out occurring where the weak beds, often shale layers, reached above the haunch area. The notable exceptions to this typical overbreak style are the Rochester and the lower Power Glen Unit, which being more homogeneous shale units, with a thickness of ~1 tunnel radius, had a more continuous longitudinal geometry. Specific observations for the various formations are described below.

Figure 5.14: Summary of the zones of behaviour observed throughout the first 3 km of the tunnel.

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Table 5.2: Overbreak observation summary for each formation in Zone 1 with minimum and maximum depth and width measurements indicated (N/M = not measured). Where overbreak was minimal or localized it is indicated by “see notes” for further details.

<table>
<thead>
<tr>
<th>Formation</th>
<th>Overbreak Notes</th>
<th>Overbreak</th>
<th>Overbreak</th>
<th>Overbreak</th>
<th>Overbreak</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Crown</td>
<td>Sidewall</td>
<td>Floor</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Depth</td>
<td>Width</td>
<td>Depth</td>
<td>Width</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(m)</td>
<td>(m)</td>
<td>(m)</td>
<td>(m)</td>
</tr>
<tr>
<td>Lockport</td>
<td>Stable</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Decew</td>
<td>Fracture growth on shale partings</td>
<td>0.15 – 0.2</td>
<td>1-2</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Rochester</td>
<td>Fracturing along gypsum infilled bedding. In haunch area, cracks across bedding</td>
<td>0.05-0.6</td>
<td>4-5</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Irondequoit</td>
<td>Feathered edge of bedding leaves a 25 mm step to the excavation geometry</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Reynales</td>
<td>Loosening focused at feathered edge of argillaceous beds</td>
<td>0.05-0.08</td>
<td>See Notes</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Neagha</td>
<td>Rock loose/broken when exposed at back of cutter head (crown and invert)</td>
<td>0.7</td>
<td>4</td>
<td>0.01-0.02</td>
<td>2</td>
</tr>
<tr>
<td>Thorold</td>
<td>Dilation on argillaceous beds, minor loosening at feathered edge of beds</td>
<td>See Notes</td>
<td>See Notes</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Grimsby</td>
<td>Overbreak focused at feathered edges of shale beds. Thicker shale beds resulted in large overbreak. Sidewall popouts on occasion. Similar to the Grimsby</td>
<td>0.05-0.15</td>
<td>See Notes</td>
<td>See Notes</td>
<td>See Notes</td>
</tr>
<tr>
<td>Power Glen Upper Unit</td>
<td>Local wedges formed by bedding and vertical joints, longest 10m. Overbreak creating a flat crown</td>
<td>0.05-0.4</td>
<td>See Notes</td>
<td>See Notes</td>
<td>See Notes</td>
</tr>
<tr>
<td>Power Glen Lower Unit</td>
<td>Dilution on bedding, minor loosening at feathered edges, focused in the crown</td>
<td>0.35-2.7</td>
<td>1.0-9.0</td>
<td>See Notes</td>
<td>See Notes</td>
</tr>
<tr>
<td>Whirlpool</td>
<td></td>
<td>0.05</td>
<td>See Notes</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
Table 5.3: Overbreak observation summary for the Queenston in overbreak Zones 2 - 4. Minimum and maximum depth and width measurements indicated and N/M = not measured. Where overbreak was minimal or localized it is indicated by “see notes” for further details.

<table>
<thead>
<tr>
<th>Overbreak Zone</th>
<th>Overbreak</th>
<th>Overbreak Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Crown</td>
<td>Sidewall</td>
</tr>
<tr>
<td></td>
<td>Depth (m)</td>
<td>Width (m)</td>
</tr>
<tr>
<td>2</td>
<td>1.0-1.4</td>
<td>6-8</td>
</tr>
<tr>
<td></td>
<td>Blocky and breaking back to the Whirlpool contact creating flat back, transitioning to variable geometry. Sidewall damage patchy and irregular</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>0.8-3.0</td>
<td>5-6</td>
</tr>
<tr>
<td></td>
<td>Variable geometry, notch changing sides and at times not forming. At apex of gorge, minimal overbreak (&lt;0.25 m). Minor sidewall damage on left side only.</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>3.0-4.0</td>
<td>7-8</td>
</tr>
<tr>
<td></td>
<td>Consistent notch geometry where spiles not installed. Breaking to underside of spiles, leaving half barrels. Minor sidewall damage on left side only.</td>
<td></td>
</tr>
</tbody>
</table>

The overbreak in the Rochester Formation was limited to the crown and invert, with no sidewall damage observed. In the crown area, high angle induced fractures cut across the bedding, creating slabs of rock and a stepped edge to the overbreak profile, as shown in Figure 5.15 a and b, respectively. In places, evidence of shear failure in the crown was observed as rock split across bedding to form slabs parallel to the excavation profile. The overbreak would break back through fracture growth parallel to the bedding, creating a flat geometry. A note of interest, water inflows from the overlying Lockport caused adverse working conditions for the tunneling crew well into the Rochester (laterally ~75 m beyond the contact) since the overbreak extended up to the contact.
Figure 5.15: a) Induced fracture cutting across bedding in the haunch area and b) stepped overbreak profile in the crown, in the Rochester Formation.

At approximately 1 radii below the Decew contact the width of the overbreak in the Rochester became relatively consistent until approximately half a radii. At this point the width decreased as the tunnel passed from the Rochester into the underlying Irondequoit, as shown in Figure 5.16.

The Neagha is a very weak, fissile shale and in the tunnel crown it broke back to the overlying Reynales Formation, Figure 5.17a. Being only 2.1 m thick, the formation was located in the crown over a short distance during the downward drive.

The Thorold Formation, with the interbedded shale layers, resulted in bed parallel fractures and minor feathered edge loosening and fall out. This is shown in Figure 5.17b. The
irregular nature of the interbedded shale layers created a more stable configuration with the more competent sandstone layers, reducing the volume of the overbreak.

The Grimsby, however, generally had laterally continuous shale beds in the range of 0.1 to 0.2 m thick. Minor loosening and bedding parallel fractures were observed to open 0.001-0.002 m in the haunch area and fall out occurred as they approached the tunnel crown, (Figure 5.17c). The competent sandstone layers minimized the depth of overbreak. Some minor stress induced failure occurred where shale bed movement caused the thin shale layer to pop out from the excavation surface in the sidewall area, as shown in Figure 5.17d.

![Overbreak Width Diagram](image)

**Figure 5.16:** Overbreak width in the Rochester Formation, showing the relationship between the width and the overlying and underlying formations at approximately 1 x radius and 0.5 x radius of the tunnel, respectively. Data courtesy of Ontario Power Generation.
Figure 5.17: a) Neagha overbreak on top of the cutterhead, b) shale interbed loosening in the Thorold, c) overbreak following shale beds in the Grimsby, d) a field notebook resting on a shale bed which has popped out 0.03 m in the Grimsby, e) haunch wedge which was removed in the Power Glen and f) bed parallel fractures in the Power Glen.
The Power Glen upper unit had minor instability and dilation of beds in the crown, with minor overbreak along shale beds, similar to the Grimsby Formation. The lower unit had overbreak in the crown area, but was of limited height vertically above the crown due to the more competent upper unit. It typically formed a flat topped geometry illustrated in Figure 5.13, Zone 1, Section AA’ and shown in Figure 5.17e.

Where encountered, gravity wedges were supported in the tunnel. At one location, a long continuous vertical joint ran 20-30° off the tunnel axis. This type of wedge was held in place in part because a third joint surface to create fall or slide out conditions was missing, and the compressive strength of the sandstone layers was high enough to prevent immediate snap through, however, bed parallel fractures still occurred, weakening the rock mass (Figure 5.17f). The rock support was installed in sufficient time to prevent complete fracture propagation through the sandstone beds that would destabilize the wedge.

The dimensions of the overbreak were largely controlled by the stiffer and stronger layers above and below those where overbreak was of significance. This is most notably demonstrated in the Rochester Formation. As the tunnel passed into the Queenston Formation other influences began to dominate the overbreak mechanism and geometry.

5.6.2.2 Overbreak in the Queenston – Zones 2 through 4

Behaviour of overbreak in the Queenston Formation is divided into three zones, as illustrated previously (Figure 5.15). Even though within each of these zones the overbreak can be generalized, there is a transition area between each zone which is difficult to determine precisely and is related to local changes in the magnitude of the stress field and other variability of the rock mass properties. The changes in the overbreak style will be highlighted to address the large ranges in dimensions presented in Table 5.2.
The contact between the Whirlpool and Queenston formations is an unconformity, marking the transition from Ordovician (Queenston) to Silurian (Whirlpool) time and the start of Zone 2. The stiffness contrast between the Whirlpool and the Queenston creates a local stress shadow below the contact, reducing the stress levels slightly. The exposure of the Queenston before the Whirlpool sandstone was deposited caused arcuate joints to develop in the top 10 m of the Queenston due to erosion and stress relief. Other very widely spaced joints appear to be developed due to stress relief, perhaps during gorge erosion. The stress, in combination with local jointing and the presence of the strong Whirlpool unit above, influenced the overbreak size and shape in Zone 2. The reduction in stress levels and induced failure created large blocks of rock failing from the crown due to slabbing, which was assisted by the jointing. At times, the blocks were very large, falling onto the equipment behind the cutter head. Figure 19a shows the largest block, approximately 20 t, resting on the drills. The overbreak was observed to break back to the overlying Whirlpool Formation to a maximum depth of 1.4 m, at which time forward spiling was used to control overbreak and advance the tunnel.

The crown and floor overbreak was characteristically a symmetrical shape about the centre line, where observed in sections of the tunnel without spiles. Close to the Whirlpool contact it had a flat geometry and as the tunnel went deeper below the contact it became asymmetrical, with variable height and width. At the transition to Zone 3, an inclined (toward the excavation face) induced fracture (spall surface) was observed and a variable overbreak notch began to develop. However, the behaviour was likely influenced by the approaching gorge.

On reaching the structural influence of St. Davids Gorge in Zone 3, overbreak reached depths in the order of 2.0 m, although was as shallow as 0.25 m at approximately the apex of the gorge. It should be noted that through most of this zone, forward spiling was used to minimize the
depth of overbreak. Vertical jointing, spaced 2 - 3 m, and horizontal and inclined shear surfaces were observed under St. Davids Gorge. The joint orientation was perpendicular to the gorge. The vertical jointing remained tight and clamped due to the stress concentrations in the crown and had minor influence on the overbreak geometry. The horizontal and inclined shear surfaces were assumed to affect the overbreak, although it was difficult to determine when in fact a sheared surface existed above the crown. The overbreak geometry remained asymmetric throughout this zone, however; it was generally inconsistent in size and shape (Figure 5.18), with variations in the maximum overbreak depth and apex angle measurements.

Floor overbreak remained similar to that in Zone 2 although the vertical jointing created zones of loosening at the feathered edge of the joint where it intersected the excavation surface (Figure 5.19b). Toward the transition to Zone 4, onion skin spall surfaces were more prominent, resulting from higher stresses. These surfaces are repeated along the invert area (Figure 5.19c).

When traced to the crown overbreak they form a dome shaped induced fracture surface which is parallel to the principal stress contours around the face of the excavation (Figure 5.14, Zone 4, Section BB’). The apex angle also began to be dominantly left sided away from the gorge proper (the width projected to depth), marking a transition from structural influence on the variability in the overbreak geometry to stress controlled overbreak.

The transition to Zone 4 was taken as the point where the crown overbreak became greater than 2 m deep (Figure 5.18), the apex angle became consistent and the chord closure measurements increased from 0.012 m to up to 0.020 m. This indicates that the regional high horizontal stresses were encountered and overbreak reached maximum depths of around 4 m (Figure 5.19d).
Figure 5.18: Overbreak (maximum depth and apex angle) and crown closure records at the transition from St. Davids Gorge (Zone 3) into the high stress field (Zone 4). Data courtesy of Ontario Power Generation.
Figure 5.19: a) a large block resting on the drill directly behind the cutter head, b) sidewall loosening at the edge of joint where it intersects the excavation surface, c) spall surfaces along the invert, d) stress notch ~3.8 m deep, e) notch running back along the tunnel alignment, and f) minor sidewall damage.
The overbreak zone was characterized by steep sides where induced tensile fracturing was observed in the haunch area and induced fracturing above the crown elevation, creating a plane dipping towards the face. The crown overbreak formed an arch 7-8 m wide with a consistent notch shape (Figure 5.19e), skewed to the left, likely indicating a high stress ratio with the major principal stress orientation slightly inclined from horizontal. Failure in the invert continued with induced onion skin spall planes, which were marked with plumose and conchoidal surfaces. Failure in the crown and invert, due to the stress concentrations, resulted firstly in the haunch rock area yielding, followed by crown and invert degradation. The failure in the crown and invert were the dominant locations of yielding, however, minor sidewall spalling and induce tensile fracture damage occurred between the haunch and spring line on the left side (Figure 5.19f), similar to the failure observed in Zone 2.

5.6.3 Discussion on Overbreak

At the Niagara Tunnel, varying degrees of overbreak have been observed. Generally, the formations in Zone 1 performed reasonably well for the 14.4 m diameter tunnel. The Rochester and the lower Power Glen were among the formations which performed the worst in Zone 1. The thickness of the Rochester is greater than one tunnel diameter and as such appears to have been influenced by the stiffer formation above and below, the Decew and the Irondequoit, respectively.

At the contact between the Decew and the Rochester a considerable volume of water was encountered in the tunnel, flowing through karst channels. The flowing water has more than likely reduced the strength of the Rochester in the contact area given that the width of the overbreak zone remained wider than the rest of the tunnel length in the Rochester. The stiffer Irondequoit reduced the horizontal displacements experienced in the Rochester, as well as reduced the amount of bed deflection at the tunnel crown, by providing a measure of support.
The Queenston Formation, on the other hand, performed poorly with overbreak reaching depths of 3-4 m in the crown, where the full impact of the high horizontal stresses was experienced. The high horizontal stress field caused stress concentrations at the tunnel crown and invert and due to the high stress to strength ratio stress, induced fractures were generated following the stress contours around the excavation face. The fracturing mechanism is associated with the horizontally bedded nature of the material, which creates anisotropic strength and stiffness. The mechanism is controlled by bed parallel displacements in the side walls and bed deflections at the tunnel crown, as illustrated in Figure 5.20a. Fractures above the tunnel crown propagated parallel to the bedding in the tunnel section. In the haunch area fractures followed the excavation surface (Figure 5.20b) cutting across the bedding, due in part to the preferred displacement direction (horizontal).

In the third dimension, as previously mentioned, these fractures can be traced to form a dome or chevron fracture pattern, similar to those observed at an underground research laboratory in the Callovo-Oxfordian Argilite in Bure, France (Ababou et al. 2011). The notch shape in the invert is not as well developed because the self-weight of the rock mass adds a confining stress, reducing the dimensions of the failed area; however the repeated stress induced planes are still present, as seen in Figure 5.20c.

Sidewall spalling damage was minimal throughout the tunnel and was mostly isolated to the left side of the advancing face. This could be caused by the horizontal intermediate principal stress, \( \sigma_2 \), being sub-parallel to the tunneling section, or rotation of the cutterhead. In the former, the localization of the tensile fractures would result from the stress flow around the tunnel face, at the corner of the face and sidewall.
Figure 5.20: Bed deflection in the crown a) illustrated using a numerical model with joint elements creating the anisotropic conditions similar to those of sedimentary rocks (modified from Perras 2009). b) Stress induced fractures above the crown dipping towards the face and at the haunch parallelling the excavation surface and c) similar behaviour in the invert with excavation parallel fractures above the invert level.
As the intermediate principal stress flows around the excavation face, a low confinement zone is created on the right hand side and high confinement zone on the left hand side. The high confinement promotes tensile fracturing parallel to the minor principal stress, $\sigma_3$, creating excavation parallel fractures. These fractures generally did not coalesce to form a large spalled area, rather they resulted in distinct surfaces 0.02-0.03 m deep, spaced 0.1-0.2 m apart in clusters, as shown in Figure 5.19f, and produced negligible overbreak, as the fractures were only in the order of 1-2 m in length and occurred as clusters at irregular intervals. These fractures could also be caused by the rotation of the cutterhead. The cutterhead spins in a clock-wise manner, creating a left-hand and downward pull. This could cause increased damage on the left hand side of the tunnel, as the cutterhead oscillates back and forth during mining cycles. Although the surface fracturing was not observed continuously throughout the Queenston, this suggests that the intermediate principal stress was playing the dominate role in the damage. As the tunnel turned into the high horizontal stress, the fracturing on the left hand side ceased, probably because the intermediate stress was again aligned parallel to the tunnel face.

Local and minor in volume, overbreak occurred where vertical joints intersect the tunnel, as shown in Figure 5.19b. The failure process here is induced by movement on the joint and loosening during the excavation process, which removes the material where the wall rock is thinnest along the joint. When the joint intersects the tunnel sidewall in the haunch area, gravity assists in loosening the material for removal due to the excavation process or subsequent fall out at a later time.

The orientation of the high horizontal stress, with respect to the tunnel alignment, and variability in rock mass properties, including strength and stiffness changes, are the dominant influences on any deviations from the observed behaviour at other locations throughout the
tunnel. The deviations in the stress field are greatest around St. Davids Gorge, where there is a complex stress flow path around the irregular gorge geometry and complex stress relief structures. A better understanding of the origin of St. Davids Gorge will be beneficial to future excavation project of a similar nature. Observations from the Niagara Tunnel Project are at least able to narrow the possibilities of the gorge’s origin from a geological perspective.

5.6.4 On the Origins of St. Davids Buried Gorge

As there is little documentation of the gorge in the literature, it is important to document the observations from the Niagara Tunnel Project. Three main features were observed, jointing, horizontal shear surfaces with up to 0.01 m clayey gouge, and inclined shear surfaces with thick clayey gouge and rubble, as shown in Figure 5.13. The writers are in agreement with Spencer’s (1907) suggestion of a preexisting shallow valley that supports the initial placement of the gorge alignment, which could be in relation to pre-existing joint networks. It is difficult to determine if the joints pre-date the gorge or have formed due to gorge erosion. The observations from the tunnel indicate repeated movement of structural features, generating gouge on the shears, which could correspond to glacial episodes in the area, as there is till infilling the gorge. Stratified sediments in the gorge would require that the gorge existed for some time to allow sufficient deposition. During this time, repeated ice surges could cause the shearing and gouge development.

The long continuous, planar joints in the upper Queenston Formation were only observed in the upper 30 m and were infilled with calcite 0.005-0.015 m in thickness. They were not observed in a test adit near the Niagara River Gorge, suggesting that there is a relationship to the St. Davids Gorge alone. The continuous planar nature and calcite infilling suggests that there was a uniform loading/unloading environment allowing for fracture creation and relaxation,
permitting calcite infilling. This would also require ground water flow to allow for precipitation of the calcite. The water could be from a river flowing through the gorge or could be from glacial melt water. The latter does promote the precipitation of calcite (Menzies 2002) and the gorge does meet other criteria of Jorgensen and Sandersen (2006) for a tunnel valley, such as undulating longitudinal profile and a non-continuously falling thalweg.

Regardless of the source for the water eroding the gorge, the direction of stress relief was towards the gorge, as indicated by the joint orientations. Once the stiff Whirlpool sandstone was eroded through, the erosion would likely have progressed rapidly due to nature of the Queenston (high swelling potential and low strength). This relief towards the gorge was accommodated by horizontal and inclined shear surfaces. The horizontal shear surfaces, well below the base of the gorge, can be explained by the stiffness contrast between the Whirlpool and Queenston formations, as well as the anisotropic stiffness of the sedimentary formations. This creates an increased horizontal component of strain over a gorge formed in an isotropic rock mass. The horizontal shear surfaces are in agreement with numerical work conducted by Perras and Diederichs (2007), which indicate a flattened bulb of shear strain existed once the gorge erosion passed below the Whirlpool cap. The vertical movement is accommodated by the inclined shear surfaces.

Little other evidence was encountered in the tunnel to distinguish between a tunnel valley and a natural valley, however, small movements on the sheared surfaces, in the order of 0.1-0.6 m (observed) do exclude the possibility that the alignment of the gorge is controlled by a fault, which would have shown larger displacements. The orientations of the structures near the St. Davids Gorge are more likely to have resulted during or after gorge erosion, rather than prior and are stress relief structures.
5.7 Conclusions

Observations from the Niagara Tunnel Project are generally in agreement with regional observations of the geology. As well, the nature of the formations and the structural trends observed during the excavation of the first 3.1 km of the tunnel agree with those from the investigation phase of the project. Four Zones of behaviour were delineated from the observations;

Zone 1: Rock above the Queenston Formation where overbreak is controlled by the strength of the rock and the influence of formation above and below that immediately in the crown.
Zone 2: Blocky overbreak near the Whirlpool contact within a stress shadow. Transitioning to a notch shaped geometry.
Zone 3: The area which is influenced by St. Davids Gorge resulting in variable overbreak geometry.
Zone 4: The area which is influenced by the regional high horizontal stresses, creating an asymmetrical overbreak geometry, with a consistent notch shape commonly observed in brittle rocks where spalling occurs.

Of the four Zones, Zone 1 is the most distinct with the overbreak associated with the formations or interbedded layers with a UCS of less than 50 MPa. The thickness of the weaker layers generally determined the depth of overbreak and the style, which can generally be described as gravity slabbing with assistance from stress induced fracture growth observed in the Rochester where the stress to strength ratio was high. The formations above and below the formation directly in the crown also influence the geometry of the overbreak area, most obvious in the Rochester Formation. Where gravity slabbing was present, it was manifested visually as bedding parallel fractures. This failure mode was prominent in Zone 1, however; Zones 2 through 4 are characteristically different.
Zones 2 through 4 can be generally summarized as having overbreak as the result of stress induced spalling. Zone 2 marks the transition from slabbing at the Whirlpool contact in the stress shadow to spalling towards Zone 3. The behaviour of Zone 3 is interrupted from the typical high stress behaviour due to the presence of St. Davids Gorge. As the tunnel passed away from the gorge the high regional stresses induce spalling, which creates the typical notch shaped overbreak geometry of Zone 4.

The large volume of overbreak in the Queenston was the leading factor contributing to delays in the project. Realizing difficult tunneling conditions, the owner and the contractor mutually agreed to change the tunnel alignment, minimizing the remaining length of the tunnel in the Queenston Formation. Once out of the Queenston, tunneling resumed at the expected advance rates and the tunnel excavation was completed on May 13th, 2011. The diversion tunnel was in full operation, after lining was completed, in March 2013.

5.8 Acknowledgements
The authors would like to thank Ontario Power Generation, Hatch Mott McDonald/Hatch Acres and Strabag for their cooperation in this work. The opportunity to assess and document the horizontally laminated ground behaviour, by the authors, provided the backdrop for a larger study of the behaviour of circular tunnels in horizontally laminated ground. The authors would also like to thank Mr. Helmut Wannenmacher for his comments on the paper, as well as those of the other reviewers.

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Chapter 6: Observed Behaviour of Excavations in Carbonate Rocks

6.1 Introduction

For many centuries, carbonate rocks have had many applications worldwide, as a building stone and in cement and steel production. In addition, they are one of the most frequently used materials for aggregate (Cargill and Shakoor 1990). Geologists typically use the term ‘carbonate’ to describe sedimentary rocks that are composed predominantly of calcium. Limestone is a more generic name used to describe CaCO$_3$-bearing rocks; similarly dolomite is used to describe CaMg(CO$_3$)$_2$-bearing rocks.

Underground construction in carbonates has largely been limited to shallow excavations in North America. These include underground quarries and mines, as well as civil engineering projects. The relatively high strength of most carbonates, with average unconfined compressive strengths (UCS) of around 100 MPa, make it an ideal material for construction, whether with, on, or in the rock.

There is, however, large variation in the strength and behaviour of carbonate rocks, both at laboratory and excavation scales, and UCS values can range from 50 to 250 MPa according to Hoek (2007). This wide variability in strength is caused by many interacting factors, including grain size variations, pore volume, cement type and pervasiveness, fossil content, non-calcareous content, burial depth, dolomitization and karstification. In addition to peak strength, the nature of the carbonate matrix, the fabric, and the grain components can control the relationship between damage initiation (onset of fracturing) and the propagation of fractures, thereby controlling the brittleness of the material, or conversely its ability to sustain damage without failure. These

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4 This chapter is in preparation for submission to an international journal and will have the following author list: Perras MA, Amann F, Wannenmacher H, Ghazvinian E, Diederichs MS.
characteristics are of importance for estimating slab resilience, aggregate durability and the long-term stability of sensitive underground works such as nuclear waste repositories.

### 6.2 Information from Current Excavations in Limestone

Mining limestone aggregate is a high-volume, low-value operation. Typically, mining of limestone is begun at the surface using open pit techniques, however as a result of increasing public pressure, underground mining of limestone is becoming more common, minimising transportation distances between the mine and major centres of demand (Shinobe 1997).

Underground mining applications in North America, from which we may gain experience, are mostly limited to the near-surface, typically less than 300 m. These operations usually employ room and pillar extraction methods, making use of the horizontal to sub-horizontal bedding of the limestone formations. In addition, most of the civil engineering projects in North America are also constructed near to the ground surface. A range of mining and civil engineering projects in North America have been reviewed to more fully understand failure mechanisms in limestone excavations.

In Europe, it is more common for civil engineering projects to be constructed at greater depths than in North America, particularly in the Alpine regions. A variety of projects from Switzerland have been investigated to broaden the understanding of the rock mass behaviour of underground excavations in limestone. The depths of these excavations in Europe are more comparable to the depths of the proposed underground nuclear waste repository in Ontario, Canada. By understanding the range of behaviours of subsurface limestone excavations, it will be possible to improve siting methods for high-level nuclear waste repositories, to narrow the search. In addition, it may be possible to predict how an excavation damage zone may develop in a repository hosted in a limestone formation.
6.2.1 North American Excavations

There are currently no underground aggregate mines in Ontario, however there are 86 active mines in the United States (MNR 2010). The National Institute for Occupational Safety and Health (NIOSH) in the United States has conducted extensive surveys of underground stone mines in order to develop guidelines for safe working conditions. Esterhuizen et al. (2008a) observed that these mines typically use room and pillar methods, in which there are two general failure scenarios: low-stress shearing on existing structures, or high-stress intact rock failure.

Esterhuizen et al. (2006) developed a rating system based on a description of pillar damage, which is related to the severity of stresses flowing through a pillar. With increasing magnitudes of stress, there is more damage observed in the pillars. The sketches and descriptions in Figure 6.1 show that as the damage becomes more severe the bedding planes have an increasing influence on the failure geometry. The increasing influence of bedding planes results in pillars with a stepped profile, as indicated in the ‘severe’ and ‘very severe’ categories of Esterhuizen et al. (2006).

The descriptions of damage severity from Esterhuizen et al. (2006), and the photos in Figure 6.1 suggest that the propagation of fractures across bedding planes causes a side-step in the fracture pathway. The hourglass shape observed under severe to very severe stress states is the result of the stress flow through the pillar; it is common in pillars of other rock types too. The stepped nature is characteristic of sedimentary rocks or metamorphic rocks with a strong foliation.
Figure 6.1: Stress severity ranking system based on observed pillar damage (after Esterhuizen et al. 2006), with representative pillar damage photographs for rankings 2–5 (from Esterhuizen et al 2008a and this study – Gonzen Mine (5)).
Esterhuizen et al (2008b) proposed an optimal pillar design, with a minimum width to height ratio of 0.8, and stress loading of an individual pillar less than 25% of the UCS. This latter criterion is interesting as it is below the typical range of the Crack Initiation (CI) threshold of intact rock, which typically ranges between 30-50% of the UCS (Brace et al. 1966, Lajtai and Dzik 1996, Pestman and Van Munster 1996). An empirical design chart plotting the pillar width to height ratio against the stress to strength ratio was presented by Esterhuizen et al. (2008a) and is shown here in Figure 6.2 with the addition of a line showing the stability limit.

![Empirical stability limit chart](image)

**Figure 6.2: Empirical stability limit based on US stone mine pillars (modified from Esterhuizen et al 2008a).**
The pillar failures are likely occurring sometime after mining in that particular area and reflects a time dependent strength aspect which is beyond the scope of the current study. In stone mines, the pillars are square or rectangular in cross section and are designed to minimize the amount of economically important aggregate left behind to support the roof. A similar practice is conducted in coal mining, and Molinda and Mark (2010) have documented the types of failures observed when mining under weak roof rock conditions.

The findings of Molinda and Mark (2010) document important failure mechanisms in sedimentary rocks related to coal mines. Their findings are summarized in Figure 6.3. Of interest from their observations are the signs of roof instability, such as central tension cracks, stairstepping tension cracks at the abutments, and roof cutters at the abutments (see Figure 6.3). Molinda and Mark (2010) also documented other failure mechanisms more closely related to coal mining, which are not discussed here. Understanding failure mechanisms is important for linking the influence of bedding planes to the failed geometry.

In civil engineering projects, subsurface excavations are more commonly long continuous tunnels, adits or rooms, where the design focuses on the maximum stable span, stable radius, and intersections with other excavations or appropriate spacing between parallel excavations.

Examples from North America are generally limestone cavities with only localised instabilities. Some notable examples are shown in Figure 6.4. These sites are generally located in limestone that has widely to very widely spaced bedding with few vertical or inclined joints or structures. In order to investigate limestone behaviour at greater depths, projects outside of North America were sought, where underground infrastructure and mining projects are more common at depths greater than 300 m.
Figure 6.3: Rock failure mechanisms in coal mines with weak roofs, showing: a) central tension crack due to bed separation and deflection; b) side stepping abutment cracking due to bed deflection; and c) abutment cutter (fracturing) due to high horizontal stresses.
Figure 6.4: Civil engineering projects constructed in limestone in the Michigan and Appalachian sedimentary basins. a) Stable limestone at the Norton Mine after conversion to a compressed air energy facility. b) Localized damage due to bedding planes at the Darlington Nuclear Power Water Intake Tunnel. c) Stable Lockport limestone at the portal to the Niagara Tunnel Project (NTP). d) Localized damage in the Irondequoit limestone at the NTP. E) Roof slabbing in the Lockport at the NTP and f) the circular profile of the TBM excavation in the Lockport at the NTP (from Hydrodynamics 2013 and Perras 2009).
6.2.2 Excavations in the Quintner Limestone

In Europe, infrastructure networks have been expanding, not only to improve transportation times between major centres within the European Union, but also to meet the hydropower demands of growing populations. Many infrastructure projects have recently been completed or are under construction within the Quintner limestone in Switzerland. A variety of projects have been reviewed in the literature, from underground factories to hydropower caverns and mines. Several of these infrastructure and mining operations have been visited as part of this research project.

Figure 6.5: Generalized tectonic units of Switzerland (modified from Swiss Topo (2005) with units from Hsu 1995). The location of sites discussed in this chapter are shown.
Figure 6.6: Thrust slices of the Alvier mountain group and the location of Gonzen Mountain near the town of Sargans, Switzerland (modified from Pfeifer et al. 1988 and Stadlin 2010).

The Quintner limestone is located in the Helvetic zone of the northeastern Alps. The major tectonic units of the Swiss Alps are shown in Figure 6.5. The overthrust units form a series of imbricated nappes, called the Helvetic Nappes (Pfiffner 1981). These nappes form the Alvier and Gonzen mountain chains, as shown in Figure 6.6. Pfiffner (1981) indicated that a transition
from imbricated thrusting to folding occurs between the Alvier Mountains in the west and Gonzen Mountain to the east, separated by the Santis Thrust (see Figure 6.6).

6.2.2.1 Underground Projects near Gonzen Mountain

Several underground projects have been completed, are in operation or are under construction in the Gonzen Mountain region. These include an underground microchip manufacturing facility, two underground quarries, two fortifications, a hydropower project, and an abandoned iron mine.

Stadlin (2010) recently examined the behaviour of several of the above underground facilities within the Quintner limestone, in the Sargans area at the base of Gonzen Mountain. Surface exposures of the Quintner limestone show topographic-parallel joints or exfoliation joints (Figure 6.7a). These joints are often filled with clay, but they allow water to flow underground and create karst cavities (Figure 6.7b). The joint aperture and infilling can be very large (20 cm wide in Figure 6.7c). Stadlin (2010) indicated that these joints only exist within 350–400 m of the thrust fault that divides the Gonzen and Schollberg nappes (Figure 6.6).

When constructing the underground excavations in Gonzen Mountain, large wedges were encountered, such as that shown in Figure 6.8a. Wedges, open joints and karst cavities created irregular roof profiles (Figure 6.8b) and can cause roof instability during excavations (Wannenmacher et al. 2010).

In addition, shear zones associated with thrusting and nappe formation create zones of weakness which can also result in excavation instability, as shown in Figures 6.8c and d. These instabilities are driven by gravity and a lack of rigid confinement, due to the clay infilling or crushed nature of the rock mass. Stadlin (2010) reports that other excavations in the Quintner limestone, such as the Flims bypass and Engelberg tunnels, also encountered challenges in the form of large wedges and water inflows from karst cavities.
Figure 6.7: Near surface characteristics of the Quintner limestone showing: a) exfoliation joints at surface; b) karst cavity; and c) 20 cm-thick joint (Stadlin 2010).
In excavations where there were few clay-filled joints, there were no roof instabilities (Stadlin 2010). With increasing depth below the ground surface, a decreasing number of joints and increasing clamping stress is encountered. A sub-horizontal access adit into the Gonzen mine proper provides an opportunity to examine this transition from gravity- to stress-driven instabilities.

Figure 6.8: Near surface excavation responses in the Quintner limestone near Sargans, Switzerland, showing: a) wedge failure along joint surfaces; b) irregular roof geometry due to blocky ground; c) roof collapse up to more competent rock above a shear zone, and d) chimney-style failure up through a shear zone (modified from Stadlin 2010 and Wannenmacher et al. 2010).
6.3 Gonzen Mine

Gonzen Mine is located within Gonzen Mountain near the city of Sargans, in the northeast of Switzerland (see inset in Figure 6.9). Folding and faulting during mountain building exposed iron ore at high elevations on Gonzen Mountain.

The Romans were likely the first to encounter the iron ore, and they began mining near the peak of Gonzen Nappe, Figure 6.9 (Hugger 1991). The long history of mining in this area has been documented by Hugger (1991), and the last stage of mining was conducted between 1919 and 1966, finally ceasing due to poor economic conditions (Pfeifer et al. 1988). Recently a portion of the mine was reopened as a tourist attraction.

6.3.1 Geological Conditions

The hematite-manganese ore body is contained within the folded sequence of the Quintner limestone. The Quintner limestone is Upper Jurassic (Malm) in age. The ore is interbedded at the base of the Platten unit, within the top of the lower Quintner unit. The Platten, a marl band, transitions into the more competent upper Quintner unit, with a total thickness of approximately 350 m (Heim 1917). The foundation of the mountain includes the Schilt and Reischiben Formations, which are not exposed in the mine. The stratigraphy is shown in Figure 6.9, in relation to the mined ore horizon.

6.3.1.1 Deposition

The sediment that makes up the Quintner limestone was deposited in relatively deep waters (100–1000 m) and is therefore predominately a dark grey biomicritic lime-mudstone. The iron ore was formed by hydrothermal venting on the sea floor (Pfeifer et al. 1988). The contact is transitional, from minor hematite inclusions away from the main ore body to a massive fabric in the main ore zone. The contact is sharp between the limestone and the massive ore (Pfeifer et al. 1988).
Figure 6.9: Stratigraphy of Gonzen Mountain with the surface of the exposed ore horizon to show its character.
Pfeifer et al. (1988) indicated that the ore’s hydrothermal origin would require reactivation of a local fault system, however, the only evidence for this are breccia beds occurring above and below the ore zone. Otherwise, the faults encountered are all a result of post-depositional tectonics.

6.3.1.2 Tectonics
The peak of Gonzen Mountain is itself an anticline with the fold axes generally striking SW-NE (Stadlin 2010). Differential movement along individual thrust surfaces was accommodated by strike-slip movement on transverse faults (see Figure 6.9) during tertiary thrusting and folding (Pfiffner 1981). This deformation weakly metamorphosed the rock formations, and calcite veins developed along bedding and within extensile fractures (Figure 6.10). The deformation events also influenced the rock mass properties and the stress field.

Figure 6.10: Calcite veins observed in a fold underground at Gonzen Mine.
6.3.2 Stress Field

A cavern in the Quintner limestone is being constructed at a similar depth to the Gonzen mine. For this project, the primary vertical to horizontal stress ratio ($K_o$) was estimated to range from 0.28 to 0.70 (Wannenmacher, personal communication). In an effort to narrow the range of stress values, an elastic topographic model of Gonzen Mountain was created. This model (Figure 6.11) ignores the influence of tectonic locked-in stresses, faulting, and other heterogeneities in the rock mass that could have a large influence on the stress. However, the results can be taken as the lower limit of the stress ratio.

Figure 6.11: Topographic elastic model along the access tunnel, at 500 m a.s.l. elevation, showing the vertical (top) and horizontal (bottom) stress contours from Phase2 by Rocscience.
The topography along the access adit was used in the elastic model to determine the stress contours as a result of the weight of the overlying rock mass (Figure 6.9). The vertical and horizontal stresses were determined to be 18.3 MPa and 6.4 MPa, respectively, at the main mine level. This results in a $K_o$ ratio of 0.35. This should reflect the lower limit of stress, since other influences on the stress field would tend to increase the horizontal stress more than the vertical. This calculation represents a starting point for a calibration based on observed failure at different locations in the mine. Field and laboratory testing were conducted to determine the physical properties of the Quintner limestone, for inputs into the numerical models.

### 6.4 Field and Laboratory Testing

Field investigations were conducted at Gonzen Mine by the author to further understand the nature of the spalling, which had been previously documented by Stadlin (2010). As part of the field investigations, Schmidt hammer readings and point load testing of samples from the mine were conducted to compare with laboratory testing results.

Schmidt hammer readings were conducted at the intersection of two adits, as shown in the inset plan views of Figure 6.12. A series of readings were taken moving away from the intersection, to determine if a change could be detected in the quality of the rock mass. The top graph, in Figure 6.12 shows that the average rebound energy, averaged from a number of readings at one location, increases around 4 m from the intersection, approximately reaching a steady value around 40%. There is a large scatter in the individual readings, however, the average trend is useful for understanding the potential damage at the excavation surface. Generally there is an increasing rebound energy moving away from the intersection. This distance roughly corresponds to the size of the spalled region, which will be discussed later in this section.
Figure 6.12: Increasing Schmidt hammer rebound energy moving away from adit intersection (see inset; top) and decreased rebound energy in areas of visible spalling (bottom).
Similarly, readings were taken around the perimeter of the adits along vertical cross sections at increasing distances from the intersection. In this adit it was observed that spalling in one of the upper corners was continuous along the length of the adit. In the bottom graph of Figure 6.12, there is clearly lower rebound energy in the areas of observed spalling. However, there is no distinguishable trend moving away from the intersection. The spalling along this adit was continuous along the axis of the adit in the observed area. The difference in the orientation between the intersecting adit, which exhibited decreased spalling moving away from the intersection, is likely related to the stress field orientation. The floor of the adit was covered with muck, so readings could not be taken, although the rebound energy decreases towards the ditch.

Schmidt hammer readings were also conducted in two caverns in the Quintner limestone, on pillars in the top headings. The pillars were being excavated using drill and blast methods. A view looking down cavern A, 160 m-long by 30 m-wide, is shown in Figure 6.13 with insets of the locations on the pillars where measurements were taken. The drill and blast excavation method likely results in a baseline amount of damage to the rocks mass. To overcome this baseline it was important to take multiple readings from the same location. The readings are less conclusive for the first pillar (Figure 6.14 top) with the average values being similar in all positions. However, the maximum readings indicated that the corner of the pillar has the lowest rebound energy. This is in agreement with the second pillar measured (Figure 6.14 bottom) where the average reading indicated that there is a lower rebound energy closer to the corner.

The Schmidt rebound is dampened more by increased damage to the rock mass at the corner of the pillars. The Schmidt hammer results are subtle, but do indicate that it may be a useful tool to indicate where the most severe damage has occurred. This may be particularly useful in areas where the damage zone is not visible.
Figure 6.13: View along the top heading of the cavern A (160 x 30 m) in the Quintner limestone, Switzerland. The inset images show the pillars and the locations of Schmidt hammer readings for: (right) cavern A; and (left) cavern B.
Figure 6.14: Schmidt hammer return energy readings, indicating greater damage to the rock mass at the corner of the pillars in the two caverns excavated in the Quintner limestone.
6.4.1 Tensile Strength

Point load measurements were conducted on samples from within the mine to estimate the tensile strength of the intact rock. The point load measurements were converted to tensile strengths using:

\[ T = -1.5I_{s(50)} \] (6.1)

where \( I_{s(50)} \) is the point load index in Equation (6.1) of Zhang (2005). The point load measurements are shown in Figure 6.15 and are compared to Brazilian Tensile Strength (BTS) values tested in the laboratory.

Tensile strength was also determined from laboratory studies, as part of the practicum program at the Geological Institute at ETH, Zurich, using Brazilian and point load tests on the samples from the field investigations in this study, as shown in Figure 6.15.

There is a consistent distribution of tensile strengths between the two methods, although the point load results appear to truncate at 8 MPa. The average value for the Brazilian tests was 6.4 MPa and that from the point load tests was 5.4 MPa. A value of 5.9 MPa, as a mean of these two results, was used in the modelling presented later in this chapter.

In some cases the results were influenced by calcite veins in the samples, resulting in lower estimates of the tensile strength. When the vein was oriented parallel to the loading direction, the tensile strength was found to be approximately 2.5 MPa in the Brazilian tests. This value was taken as the tensile strength of the joint elements, to represent weakness planes in some of the numerical models.

6.4.2 Unconfined Compressive Strength

To better understand the influence of calcite veins on fracture initiation and propagation in the Quintner limestone, a set of 15 unconfined compressive tests were conducted (see Table 6.1).
Examples of the sample character are shown in Figure 6.16. The vein angles reported in Table 6.1 are for the dominate vein orientation.

6.4.2.1 Equipment and Procedures
The QU samples were subjected to Uniaxial Compressive testing using an MTS loading frame, and an axial deformation control was used at a rate of 0.1 mm/min. The samples were unjacketed and impermeable loading platens were used.

Figure 6.15: Tensile strengths estimated from point load test results, compared with Brazilian Tensile results.
Table 6.1: Quintner limestone sample dimensions and vein character (from Perras et al 2012b).

<table>
<thead>
<tr>
<th>Sample Number</th>
<th>Diameter (mm)</th>
<th>Height / Diameter</th>
<th>Vein Angle (°)</th>
<th>Vein Character</th>
</tr>
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<tr>
<td>QU-1U</td>
<td>62.90</td>
<td>2.22</td>
<td>15</td>
<td>one thin</td>
</tr>
<tr>
<td>QU-2U</td>
<td>62.90</td>
<td>2.06</td>
<td>27</td>
<td>ubiquitous, thin</td>
</tr>
<tr>
<td>QU-3U</td>
<td>62.93</td>
<td>2.42</td>
<td>-</td>
<td>no veins</td>
</tr>
<tr>
<td>QU-4U</td>
<td>62.93</td>
<td>2.30</td>
<td>42</td>
<td>several, thin</td>
</tr>
<tr>
<td>QU-5U</td>
<td>62.97</td>
<td>2.20</td>
<td>8</td>
<td>one thin</td>
</tr>
<tr>
<td>QU-6U</td>
<td>62.91</td>
<td>2.46</td>
<td>38</td>
<td>several, thin</td>
</tr>
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<td>QU-7U</td>
<td>62.93</td>
<td>2.23</td>
<td>40</td>
<td>one thick</td>
</tr>
<tr>
<td>QU-8U</td>
<td>82.76</td>
<td>2.51</td>
<td>-</td>
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</tr>
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</tr>
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<td>50</td>
<td>ubiquitous, thin</td>
</tr>
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<td>22729</td>
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<td>-</td>
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</tbody>
</table>

Axial strain was measured by a Linear Variable Differential Transformer (LVDT) inside the machine and lateral strains were measured using HBM, strain gauges, 2 cm-long, (type 20/120 LY41-3L-3M) affixed to the samples with Z70 adhesive.

Wave energy from crack initiation events was measured using Physical Acoustics Corporation NANO30 sensors with a sensitivity range between 0.1 and 1.00 MHz. Eight sensors were placed at two elevations on the samples, with four sensors equally spaced around the circumference at each elevation. Signals were detected with an ESG Hyperion system, at
approximately 3 events per second. Pre-amplification of 40 dB was applied prior to acquisition. No post-processing of the AE waveforms was conducted as only the hit count was used in this study. The ‘22’ series samples were subjected to Uniaxial Compressive testing using a modified Walter & Bai rock testing machine with a constant circumferential displacement rate of 0.01 mm/min. During the test, the axial strain and circumferential displacement was monitored using two axial strain gauges (e.g. base length of 50 mm) and a single strain gauge mounted on a chain wrapped around the samples (at 50% of the sample height). Since measuring the circumferential displacement with a chain wrapped around a sample is prone to error, the monitoring system was calibrated with a fused quartz sample.

Figure 6.16: Representative samples tested in this study. a) sample QU-6U with minimal calcite veining and 1–2mm wide styolites, b) sample QU-7U with wide, extensive calcite veining, c) sample QU-8U, which is dominantly calcite vein material, and d) sample QU-9U with fine ubiquitous veining (from Perras et al 2012b).
6.4.2.2 Strength Thresholds

The ISRM (1979) suggested methods for UCS testing and to determine the peak strength or UCS, the axial strain $\varepsilon_a$, the circumferential strain $\varepsilon_n$, and the average modulus $E_{\text{avg}}$ were used by both laboratories. To date, there are several methods for determining crack initiation (CI) and crack propagation (CD) and both the acoustic emission (AE) and strain methods were used on specific samples presented in this paper. Diederichs (2007) developed a method to use CI and CD thresholds to determine brittle parameters that may be used in numerical modelling programs that only allow peak and residual properties to be entered. Since brittle spalling was observed at Gonzen mine, it is important to determine the strength thresholds for numerical model input. In the discussion below, sample QU-8U is considered separately, as this sample is almost pure calcite and represents the material making up the veins that cut the other samples at oblique angles to the loading axes.

The stress-strain curves shown in Figure 6.17 represent the end-member behaviours from the tests (samples 22729 and QU-1U) and several other results representing the remainder of the samples. Sample 22729 is the only one that shows linear elastic behaviour at low axial stress levels. The other samples begin to behave in a linear elastic manner at stresses between 20 and 40 MPa, at least 10 - 20 MPa below CI. This behaviour is typical for samples that are damaged prior to testing, as these cracks will close in the early stages of loading. For this reason, the modulus of the samples is calculated as the average modulus from the region in which the sample behaves in a linear elastic manner.
Figure 6.17: Stress-strain responses for selected test results (out of 15 tested) of the Quintner limestone showing the variability in the sample set. The inset photo shows the arrangement of the 4 AE sensors (two top-half and two bottom-half) and 4 strain gauges (two lateral and two axial) (modified from Perras et al 2012b).

The Young’s Modulus, $E_s$, was taken over a region that typically included CI, but was below CD. On average, the modulus of the samples was found to be 52 GPa, or 58 GPa if the low modulus samples are excluded. The minimum modulus was found to be 22 GPa and the maximum was 73 GPa, as shown in Table 6.2. The calcite sample (QU-8U) is much stiffer than the other samples, with an average Young’s Modulus of 119 GPa. This sample also has a much smoother peak stress-strain response than the other samples. The other samples have irregularities
in the stress-strain response, both pre- and post-peak. These irregularities are due to the heterogeneous nature of the samples. In addition, there is a less abrupt change in the AE response after CD for the calcite sample, compared to the other samples (to be discussed later).

The compressive strength results also show a considerable range of values, between 53 and 143 MPa, with an average of 102 MPa. Five of the samples have considerably lower strengths, and if these are excluded the average strength increases to 112 MPa, which is a small increase considering the range of values. The peak strength has been widely used in numerical methods, and more recently the CI and CD thresholds have been utilized in modelling brittle rock behaviour, following the methodology of Diederichs (2007).

**Table 6.2: Strength threshold and modulus results (from Perras et al 2012b).**

<table>
<thead>
<tr>
<th>Sample Number</th>
<th>CI (MPa)</th>
<th>CD (MPa)</th>
<th>UCS (MPa)</th>
<th>E (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>QU-1U</td>
<td>35</td>
<td>46</td>
<td>53</td>
<td>26</td>
</tr>
<tr>
<td>QU-2U</td>
<td>34</td>
<td>65</td>
<td>99</td>
<td>50</td>
</tr>
<tr>
<td>QU-3U</td>
<td>48</td>
<td>100</td>
<td>119</td>
<td>68</td>
</tr>
<tr>
<td>QU-4U</td>
<td>52</td>
<td>103</td>
<td>143</td>
<td>55</td>
</tr>
<tr>
<td>QU-5U</td>
<td>38</td>
<td>54</td>
<td>63</td>
<td>29</td>
</tr>
<tr>
<td>QU-6U</td>
<td>36</td>
<td>43</td>
<td>55</td>
<td>22</td>
</tr>
<tr>
<td>QU-7U</td>
<td>58</td>
<td>105</td>
<td>134</td>
<td>63</td>
</tr>
<tr>
<td>QU-8U</td>
<td>67</td>
<td>88</td>
<td>124</td>
<td>119</td>
</tr>
<tr>
<td>QU-9U</td>
<td>43</td>
<td>54</td>
<td>63</td>
<td>73</td>
</tr>
<tr>
<td>QU-10U</td>
<td>40</td>
<td>63</td>
<td>68</td>
<td>59</td>
</tr>
<tr>
<td>22729</td>
<td>42</td>
<td>76</td>
<td>83</td>
<td>72</td>
</tr>
<tr>
<td>22730A</td>
<td>52</td>
<td>104</td>
<td>137</td>
<td>56</td>
</tr>
<tr>
<td>22730B</td>
<td>53</td>
<td>99</td>
<td>108</td>
<td>53</td>
</tr>
<tr>
<td>22731</td>
<td>40</td>
<td>67</td>
<td>76</td>
<td>50</td>
</tr>
<tr>
<td>22732</td>
<td>53</td>
<td>102</td>
<td>107</td>
<td>56</td>
</tr>
</tbody>
</table>
The CI and CD thresholds were calculated in two ways. For the QU series samples, AE methods were used, and for the ‘22’ series samples, volumetric and crack volumetric strain reversal points were used.

Acoustic emissions are generated either when grains in the sample slide along each other, when new cracks are created, or when cracks propagate (Mogi 2007). The process generates an elastic strain wave, from the sudden release of elastic strain energy, which is detected by the AE sensor when the amplitude of the signal exceeds the trigger level. The AE events were recorded during the testing when the amplitude of the wave energy reached the trigger threshold of 100 mV. The waveform was captured for a duration of 409 μs, including 90 μs prior to triggering.

The AE events were used to determine the strength thresholds, CI and CD for the QU series samples. CI was determined as the point on the stress-AE hit count graph where a significant number of events began to occur in a continuous manner, and CD was taken as the point at which there is a rapid increase in the hit count, as shown in Figure 6.18. The range of CI stresses, based on the AE results, is between 34 MPa and 52 MPa, with an average of 41 MPa. CD has a wider range: 43–105 MPa, with an average of 71 MPa. The width of the range of CD is comparable to that for UCS.

For the strain-based determination of CI and CD, volumetric strain is often used. Brace et al (1966) stated that CI is determined to be the point at which the volumetric strain, εVOL, curve departs from linearity. For samples that have been pre-damaged, the crack-closure and CI limits may overlap. Martin (1999) and Diederichs and Martin (2010) suggested that the reversal point of the crack volumetric strain, εCV, (inelastic strain) be used as the CI limit. The crack volumetric strain is determined by subtracting the elastic volumetric strain, εEV, from the volumetric strain. An example is shown in Figure 6.19 for sample 22730A.
Figure 6.18: AE cumulative crack count and CI and CD thresholds for samples QU-1U, QU-7U and QU-8U (from Perras et al. 2012b).

The CD threshold occurs at the onset of crack interaction and results in increased lateral strain over axial strain. This causes a reversal in the volumetric strain curve, and this point is taken as CD, as shown in Figure 6.19. The peak values are stated in Table 6.2. For the strain-based method, the CI range is 40–53 MPa, narrower than that found by the AE method, and a higher average of 48 MPa. A similar trend is observed for CD and UCS. When the CI and CD thresholds are plotted against the UCS values, in Figure 6.20, there appears to be good correlation between the values determined from the strain method and the AE method, in terms of the percentage of the UCS value. A discussion of the influence of veins on the laboratory testing
results follows in the next section. The wide range in values is in part due to the presence of veins in the rock.

6.4.2.3 Summary of Laboratory Findings

Peak strength and the modulus of rock have long been used in engineering design. CI and CD thresholds have only more recently become an important parameter for numerical modelling of brittle materials, despite having been recognized in laboratory data for some time (Bieniawski 1967). These parameters are now beginning to be used for engineering design projects.

Figure 6.19: An example of CI determination from sample 22730A, using the reversal point of $\varepsilon_{CV}$ (from Perras et al. 2012b).
Ghazvinian (2010) noted that the strain-based method from granite test results showed a wide range of CD values, due to a plateau in the reversal point of the volumetric strain curve. Amann et al. (2011) identified two modes of volumetric strain behaviour in the Opalinus clayshale: (1) where the curve changes sharply and CD can be determined over a short interval (2–5 MPa); and (2) where the curve changes smoothly resulting in a wider range of CD (10–20 MPa). In the current study, the strain reversal points result in a typical range of 10 MPa for both CI and CD, for those samples with volumetric strain data. As shown in the results, the AE method presented here gives a lower boundary and the strain method an upper bound. Results presented
by Amann et al. (2011) and Ghazvininian (2010) are generally similar, although the Opalinus clayshale (Amann et al. 2011) appears to have a lower AE threshold in only half of the cases.

The variability in the test results presented here could be attributed to the calcite veining, which was distributed differently in each of the samples tested. During the formation of the calcite veins, the newly created extensile fractures, resulting from tectonic deformation, promote the growth of calcite. The veins therefore should have a reduced number of flaws or pores because the calcite will fill these voids. This is supported by the test data, as the calcite-rich sample (QU-8U) was both stiffer than and almost as strong as the strongest sample tested. Therefore, the more calcite veining present in the sample, the stronger the sample should be, because micro-cracks will be arrested at vein-wall rock contacts. This leads to a non-linear stress-strain response, similar to a bedded sample.

During uniaxial compression testing, cracks first propagate parallel to the loading direction since there is no lateral confining pressure and this is the least resistant orientation (e.g. Tapponier and Brace 1976). As the cracks propagate they begin to interact and can deviate from being parallel to the loading directions. Typically, however, the failure mode is axial splitting.

The calcite veins, in the testing presented in this paper, were typically orientated between 8 and 55 degrees to the loading axis. At a low angle the strength is lower because the cracks can more easily propagate along the vein-wall rock interface (Figure 6.21). Even in the data from only a few samples, the stiffness appears to increase as the angle increases, however, the strength thresholds are less clearly distinguished.

There appears to be a very small increase in the CI threshold with increasing angle. This increase cannot be fully attributed to the veins, however, as it is within the range of reported CI values for other rock types mentioned earlier. When examining Figure 6.21, CD and UCS appear
to have no clear relation to the vein angle. Yet there is evidence that the veins influence the propagation direction of fractures during testing, as shown in Figure 6.22. The fracture side-steps and follows the orientation of the veins within the sample. A similar phenomenon was observed by Amann et al. (2011) in samples of the Opalinus clayshale, where fractures side-step due to bedding planes. The influence of veins in the Quintner limestone has also been observed at the excavation scale within Gonzen Mine, impacting the geometry of the damage zone.

Figure 6.21: Graph showing the relationship between strength threshold and the angle of the calcite vein from the loading direction (Perras et al. 2012b).
Figure 6.22: Vertical fracture offset following the orientation of veins within a UCS sample of the Quintner limestone.

6.5 Field Observations

Stadlin (2010) observed that spalling began to occur around pillars, tunnel intersections and other stress raisers approximately 1750 m into the mine along a near horizontal access adit, as shown in Figure 6.23. This corresponds to depths between 600 and 700 m below the ground surface. Below this depth threshold, little failure was observed, with an absence of joints or karst cavities. Within
the first 500–600 m along the adit from the entrance, jointing was more prevalent and resulted in blocky ground and karst cavities.

Observations were conducted at the mine by the author over the last several years, building on the work of Stadlin (2010). Within the mine proper (excluding the entrance adit) the geomechanical behaviour can be related to three main areas: pillar and intersection stress-induced failure, roof collapse in room and pillar areas, and inclined stope collapse.

There appear to be more recent failures in all three modes, which can be easily identified by the fresh grey or red surfaces that distinctly contrast with the reddish brown dust coating from the mining process, as seen in Figure 6.24. This inference is supported by accounts of red dust from the mine behaving in a similar manner to coal dust (Hugger 1994).

Figure 6.23: Summary of geological observations and excavation behaviour along the entrance adit to Gonzen Mine (modified from Stadlin 2010).
The pillar instabilities ranged from thin sheets of spalled rock on a largely intact pillar to complete crushing (Figure 6.25). Observations indicate that, in the section of the mine studied, the average room width is approximately 7.7 m with 2.2 m wide pillars between.

Figure 6.24: Spalling in the corner of a cross pillar in an inclined stope (inset). Note the contrasting colours of the spalled area (dark grey) and the surrounding rock mass (reddish-brown).
Figure 6.25: Observations of pillar failure from Gonzen Mine, showing estimated original heights and depth of failure at the mid-point.
The minimum pillar width is estimated to have been originally 1.2 m, shown in Figure 6.25 (lower left), in which failure is seen past the mid-point. The maximum depth of pillar failure observed was 1.1 m (Figure 6.25 lower right). Measuring surveyed room and pillar widths from a mine plan drawing, compiled by Swisscom in 2009, indicated average widths of 8 m and 2 m, respectively, which is in agreement with the observations. These measurements likely represent the in-situ conditions at the time of surveying and not the original constructed dimensions. The pillar stability was used as the first calibration geometry for the stress state, as this was the most common failure observed.

Intersections represent a unique opportunity to determine the influence of the distance between excavations on spalling behaviour. The distance from the intersection where spalling terminated, shown in Figure 6.26, was 3 m. The spalled region was approximately 0.5–0.8 m thick and was composed of multiple slabs, as shown in the bottom of Figure 6.26.

In the room and pillar area observed, roof collapse was isolated to one location, where the span of the room was greater than 12 m. The roof collapse is a post mining failure, as indicated by the fresh dark grey surfaces that contrast with the reddish brown dust coatings at the top edge of Figure 6.27. Similarly a post-mining collapse in a steeply inclined stope was observed (Figure 6.28). The main stope area, between cross pillars at the base, was measured to be 10.3 m wide and 13.0 m high. The contrast between the fresh spall surfaces and the intact stope walls can be clearly seen in Figure 6.28. These four observed areas will be used for back analysis.

6.6 Back Analysis of Rock Mass Behaviour

In order to better understand the failure mechanisms, empirical and numerical methods were employed. The empirical depth of spall failure and empirical stope stability can provide information about the stress field and the properties that influence the failure.
Figure 6.26: Spalling at a 60 degree intersection (top) of equally sized 2x2 m adits, with the extent of spalling diminishing away from the intersection (bottom left) and the depth of the damage (scale is 0.2 m, bottom right).
Figure 6.27: Roof collapse with slabs roughly 0.5 m thick, spanning approximately 20 m.

Figure 6.28: Stope collapse on the 505 level of Gonzen mine.
6.6.1 Empirical Stability Analysis

The empirical depth of spalling chart from Martin (1999), which was converted to be applicable to CI by Diederichs (2007), can be used to determine the range of maximum tangential stress. The range of observed spalling depths at Gonzen was 0.2–0.8 m. Since the elastic modelling provides a lower boundary for the stress ratio, the maximum depth of spalling is used with the empirical chart to determine an upper stress ratio boundary. The equivalent radius was determined as a circle inscribing a 2 x 2 m excavation. Using the minimum and maximum CI values, the range of \( \sigma_{\text{max}} \) is 32–85.3 MPa from Figure 6.28. The maximum stress is assumed to be the vertical stress and therefore \( \sigma_{\text{max}} = 3 \sigma_v - \sigma_H \). The depth is 675 m, so the vertical stress is 17.6 MPa, using 0.026 MPa/m. Since \( 3 \sigma_v = 52.7 \) and \( \sigma_H = 3 \sigma_v - \sigma_{\text{max}} \) than \( \sigma_{\text{max}} < 52.7 \) MPa. Therefore, solving for \( \sigma_H \), the range is between 2.7 and 20.7 MPa and therefore \( K_o \) ranges between 0.15 and 1.17. Since the elastic model predicts a \( K_o \) ratio of 0.35, the range of \( K_o \) used in the numerical modelling is 0.35 to 1.17.

The empirical stope stability chart of Potvin (1988) can be used to determine if the stopes at Gonzen would be expected to have stable configuration. With a hydraulic radius ranging between 3 and 6 and a modified stability number ranging between 4 and 65 (extreme values), only the maximum stope dimensions fall into the transition zone between stable and caving (Figure 6.29).

Due to the stable scenario of the stope based on the empirical design chart in Figure 6.29, other factors must have affected the stability of the stope post-mining operations. A post-mining collapse may possibly be caused by stress corrosion, creep on planes of weakness, or the combination of seismic activity with differential strains on existing faults. Using the findings of these empirical results, more detailed numerical analysis was conducted.
Figure 6.29: Empirical stability chart for unsupported open stopes, showing average (square) and extreme limits (rectangle) for the stope at Gonzen mine that collapsed. From the open stope database from Potvin (1988) and Nickson (1992), (modified from Hutchinson and Diederichs 1996).
6.6.2 Numerical Analysis

Models were constructed to simulate the various observations. The models use the average properties, outline in Table 6.3, in combination with the brittle approach of Diederichs (2007). The residual rock mass parameter, $m_r$, was set to 10. The stress field was first calibrated using the pillar observations. A slightly inclined, 10° from horizontal, room and pillar arrangement, in which the majority of the pillar observations were conducted, was used to determine the stress state that gave rise to yielding similar to that which was observed. The $K_o$ ratio was varied from 0.35, the topographic model ratio, up to 1.17. As roof failure was only locally observed, it was used as the limiting factor to calibrate the stresses.

Minimal roof yielding was found to occur for a $K_o$ value between 0.8 and 0.9, demonstrating that the topographic model underestimated the magnitude of horizontal stress. Out-of-plane stress also has an influence: when it is larger than the horizontal in-plane stress, slightly better results are obtained.

The calibrated stress field used was $K_{hv} = 0.8$ (in-plane) and $K_{Hv} = 0.9$. The pillar stability model is shown in Figure 6.30, with depths of failure indicated by the de-stressed contour (black line at $\sigma_3 = 1$ MPa) and tensile yielding, which, for example, is 0.8 m for the wider pillars. The dimensions of failure in the model are in good agreement with the observations, which showed that pillars around 2.2 m wide had an average depth of failure of 0.7 m.

The same stress field was used for a series of models representing the intersection observations. Two adits, 2 x 2 m square, were modelled at increasing distances from one another. Plastic yielding in the models was found to cease at a distance of 2.0 m between excavations. This corresponds to 2.0 m away from the intersection and the point at which the spall slab height decreases significantly, as shown in Figure 6.31.
Table 6.3: Properties of the Quintner Limestone. CI (crack initiation), CD (crack propagation), UCS (unconfined compressive strength), and $E_{50}$ from Perras et al. 2012. Tensile strength ($T$) estimated from point load and Brazilian test results, and $m_i$ is estimated.

<table>
<thead>
<tr>
<th></th>
<th>CI (MPa)</th>
<th>CD (MPa)</th>
<th>UCS (MPa)</th>
<th>$T$ (MPa)</th>
<th>$E_{50}$ (GPa)</th>
<th>$m_i$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average</td>
<td>43.6</td>
<td>77.4</td>
<td>112.0</td>
<td>5.9</td>
<td>52.3</td>
<td>12</td>
</tr>
<tr>
<td>Minimum</td>
<td>32.0</td>
<td>43.0</td>
<td>53.1</td>
<td>0.6</td>
<td>22.0</td>
<td>10</td>
</tr>
<tr>
<td>Maximum</td>
<td>67.0</td>
<td>105.0</td>
<td>143.2</td>
<td>10.4</td>
<td>73.0</td>
<td>15</td>
</tr>
</tbody>
</table>

Figure 6.30: DISL model results from Phase2, by Rocscience, for $K_{Hv} = 0.8$ (in-plane) and $K_{hv} = 0.9$ (out-of-plane), in which roof yielding is minimised ($L =$ length in meters).
Figure 6.31: DISL models, in Phase 2 by Rocscience, showing increasing distance between the 2x2 m adits to represent the adit intersection, with decreasing yield and increasing stress flow ($\sigma_3$) between the adits. The sketch shows the observed spalling, moving away from the intersection at the right (A is closest to the intersection and C is furthest away), with detached slabs (grey), spall fractures (solid black lines), and calcite veins (dashed lines).

Figure 6.32: Roof collapse models showing homogenous (top) and bedded models (bottom), with 3 m excavation stages and a 21 m excavated span.
The spalled slabs were observed to arrest along or close to calcite veins, which was not captured by the models. More detailed analysis could more accurately capture the influence of calcite veins on the yielding and therefore more accurately capture the observed geometry. However, the changes in spalling with distance between the adits agrees with the observations.

The roof collapse was a localised observation in an area that had a large span and, according to the mine plan drawing, was close to a fault. A homogenous continuum model, with the calibrated stresses, shows little roof yielding, with localized tensile yielding only in the corners between excavations with spans of 9 m (Figure 6.32 top). For larger spans, the yield extends further up, reaching a maximum depth of over 0.75 m with a span of 21 m.

In the roof collapse area, bedding was observed to be parallel to the roof, which may have acted as planes of weakness. The bedding can be included numerically using joint elements. The stiffness of the rock simulated between the joint elements must compensate for the normal stiffness of the joint element, such that the stiffness of the system is comparable to the homogenous case, using methods such as those suggested by Brady & Brown (2006). The joint properties were taken as the Mohr-Coulomb equivalent envelope, generated by the brittle parameters, derived from the minimum values reported in Table 6.3. A residual friction angle of 15° was used and cohesion and tension were set to zero.

The joints minimize the depth of yielding (Figure 6.32 bottom) by absorbing strains and focusing the stresses along the first beam. This model demonstrates the influence of bedding in reducing the depth of yielding, however widespread yielding is absent. This is consistent with the localised collapse observed, since the observed failure was possibly influenced by a more complex interaction with a fault in the vicinity and other factors controlling the local stress field.
The stope model (Figure 6.33), with the calibrated stresses, indicates a region of yield in the area where spalling was observed, at the toe of the stope (Figure 6.28). The observed spall was, at its maximum extent, 9 m high and 1.3m deep, and the model indicates yielding roughly 4 m high and 0.9 m deep. In general, the numerical yielding occurs around the access adits to a greater extent than was observed. In order to induce plastic yielding, joint elements were used and it was necessary to drop the cohesion to 2 MPa, below that used for the roof collapse model.

Figure 6.33: Stope model with inclined bedding dipping at 80°. Inset shows a close up of yield at the toe of the stope.
Despite the inadequate height of yielding simulated in the models, the depth is close to that observed. However, in this case, the depth is controlled by the presence, angle, and spacing of the bedding. A complete sensitivity analysis of the rock mass properties, joint element angle and properties is required. A more complex stress field and structural influence, as well as stress corrosion, could make up the real scenario in-situ, and this has not been adequately captured in this numerical study.

6.7 Discussion

Long-term behaviour of excavations is often thought to be limited to creep, consolidation, chemical or mechanical swelling, visco-elastic or visco-plastic behaviour. Brittle failure is often thought to be a short-term phenomenon that occurs shortly after excavation has taken place, within hours or days. However, even in the strong granites at Canada’s Underground Research Laboratory, the brittle spalling notch deepened over the course of five months (Martin 1997). The observations from Gonzen mine also suggest that brittle behaviour has a time dependent component.

The observed pillars in Gonzen show that some brittle failure occurred while mining was still ongoing, since some of the hourglass-shaped pillars are coated in the reddish-brown dust (see Figure 6.23 and 6.24). Other pillars show the dark grey colour of the limestone, which has been exposed due to thin brittle slabs failing off. Since mining ceased in the late 1960s, the continued failure of the pillars must be related to a reduction in strength over time. A conceptual model of the reduced strength envelopes with time is shown in Figure 5.34. This model suggests that as time progresses the amount of stress the rock mass can withstand near the excavation boundary diminishes. The cause of this strength reduction may be related to changes in humidity, fluctuations in the ground water table, fluctuations in temperature, or other mechanisms of
fatigue. All of these factors influence the rock at the molecular level and under steady mechanical loading can result in crack propagation, which is known as stress corrosion (Charles 1959).

However, under confined conditions, away from the excavation boundary, crack propagation due to stress corrosion should be suppressed. With the exception of stope failure, the typical spalling observed resulted in characteristically thin sheets of rock. This process might be suppressed with a thin coat of shotcrete to provide a small amount of confinement at the excavation surface. Overall, over a period of at least 50 years of exposure, the deterioration of the rock mass has generally only resulted in localised failures.

Figure 6.34: Conceptual change in the brittle failure envelope with time.
From past case histories and observations by the authors there are four general types of limestone which display different behaviour: massive, karstic, bedded, and blocky. Each will behave differently under low or high stresses. These rock mass behaviours have been summarised in Figure 6.35. Under low stress conditions, massive limestone, where bedding is spaced at distances greater than the radius of the excavation, the behaviour will be governed by localised joints, which may form wedges. At higher confinements these wedges may become more stable due to clamping, and failure will be associated with crushing the intact rock or sliding on local discontinuities. As the bedding spacing decreases it becomes a critical component of the failure mechanism. Under low stress conditions, the failure will be associated with deflection of rock beams into the excavation. This will characteristically occur across the full span of the excavation under low confinement. As the stresses increase, brittle spalling may begin to interact with the beam deflection, causing a notch to form, which will stabilise after a certain depth due to stress concentrations at the notch tip. As the rock mass becomes more blocky, ravelling can occur under low stress conditions. At higher stress conditions, sliding on the discontinuities must overcome the clamping forces and crushing may also occur. Karstic conditions can occur close to the ground surface and at depth. Under both low and high stress conditions, water inflows are a major concern in this rock type. At depth, care must be taken to reduce the risk of high-pressure inflow from the intersection with karst features.

Massive or thickly bedded limestone units with moderate stress levels will be the most stable for underground construction. A limestone with high argillaceous content will have the added advantage of crack propagation being arrested along interbeds and potential for self-sealing. The combined characteristics of good excavation behaviour and advantages of argillaceous content make such limestones ideal for underground nuclear waste repositories.
Figure 6.35: Generalized excavation behaviour in limestone.
Blocky ground conditions may be stable at elevated stress levels, by way of Vousoir arching, but the joint network would provide an unfavourable water flow pathway. Similarly, karst can provide enhanced flow through the rock mass. A study of the sedimentary rocks of Ontario shows that, below a depth of 400 m, there is no evidence for karst (Worthington 2011).

Understanding and minimising both the short- and long-term behaviour of excavations in limestone will also benefit infrastructure and mining projects and be useful for rock support design. Numerical tools for designing rock excavations and predicting dimensions of damage also benefit from the summary of behaviour presented in this research.

6.8 Conclusions
A review of existing observations from excavations in limestone reveals a wide spectrum of behaviour depending on the spacing of bedding, the strength of the intact rock, the stress magnitude, and the stress orientation. As with mudrocks, the orientation of the stress field with respect to the bedding can negatively impact the failure behaviour (when parallel with bedding) or can stabilize the excavation by increasing the normal stress on the bedding. Field observations by the authors in excavations in the Quintner limestone confirm the above behaviour. In order to understand the failure mechanisms better and the influencing factors on crack propagation, field observations, as well as field and laboratory measurements were conducted.

The field observations indicated that as the depth of overburden increased the failure mechanism changed from gravity driven to stress driven failure. Spalling was observed at areas where the stresses were concentrated and the geometry of the failed zone was in some cases influenced by bedding and calcite veining. Schmidt hammer measurements indicated that the rock surrounding the visible spalled areas is also damaged, as the rebound energy is reduced. This could potentially be a useful tool for field verification of the excavation damage zone (EDZ).
dimensions, particularly if visible damage is not present. It can also be used to verify the general stress orientation when the magnitude of the stress field is capable of causing micro-damage. The areas of micro-damage can used to verify the stress orientation.

To investigate the influence of calcite veining UCS samples were tested with varying degrees and orientations of calcite veining to determine the influence on the strength thresholds. Strength thresholds were determined using the AE method on ten samples and strain methods on five. The AE method on average gives lower strength thresholds than the strain based methods. Low variability in CI was observed for all samples and high variability for CD and UCS. The stiffness appears to be influenced the most by the vein angle. It has also been observed that the veins can alter the crack propagation direction.

The field and laboratory results were applied to a back analysis of the observed failure mechanisms at Gonzen mine. The numerical modelling presented here demonstrates the potential range of stress ratios, $K_o = 0.8–0.9$, that gives rise to yielding in the models that is similar to the observations. The pillar stability observations showed the most consistent behaviour throughout the mine, which was similarly captured in the numerical models. Observations, the empirical evaluation of stope stability, and the inability of the numerical models to capture the observed geometry of the failed zone, suggest that the long-term behaviour of the Quintner limestone has a time dependent component to the brittle spalling. The above work in combination with observations from other excavations in limestone suggest that there are four main types of limestone: massive, karstic, bedded, and blocky. Each type will behave differently under low or high stresses. Where a damage zone is formed, it will be controlled by the bedding, veins or other structures of the rock mass. This work has provided an opportunity to study short- and long-term behaviour of excavations in limestone, which in Canada is a potential host rock for nuclear waste.
6.9 References


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Chapter 7: Understanding Excavation Damage Zones in Sedimentary Rocks

7.1 Introduction

In 1957, the National Academy of Science, in the United States, stated that the most promising storage option for radioactive waste was in salt deposits (US Department of Energy 2000). Investigations into storage of nuclear waste in salt formations have predominately been conducted by the US and Germany. Research into argillaceous sedimentary rocks began in the early 1980s at select sites, such as in Belgium and Germany, however the majority of research into argillaceous sedimentary rocks began in the early to mid-1990s, as shown in Table 1.1 of Chapter 1.

In the US, the Waste Isolation Pilot Plant (WIPP) site has been a licensed facility since March 1999. Germany started investigations at the Gorleben salt dome in the 1970s (Lidskog and Andersson 2001). The attractiveness of a salt formation is that there is little water as well as good heat conduction and self-healing capacity (Langer 1999). Long-term stability of caverns in salt formations, where creep can continue to expand the Excavation Influence Zone (EIZ), is more of a concern for the safe storage of nuclear waste than in crystalline rocks where the long-term EIZ should be stable. Although fracture pathways are expected to self-heal with creep, new pathways may open or an expanding EIZ may intersect other unfavorable geological features, such as a fault. Back filling will minimize the amount of closure and therefore the expansion of the EIZ. In Canada however, one of the design criteria for the Low and Intermediate Level Waste (L&ILW) repository is the absence of backfill so that the waste can be retrievable.

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5 This chapter is in preparation for publication in an international journal and will have the following author list: Perras MA, Diederichs MS.
7.1.1 Excavation Damage Development

The geological setting and material properties of sedimentary Underground Research Laboratories (URLs) span a wide spectrum. One aspect which is characteristic of most sedimentary rocks is anisotropic strength and stiffness. This alters the way the stresses flow around an excavation when compared to an equivalent isotropic rock (Perras 2009). The anisotropic behaviour is variable and some rock formations behave more anisotropically than others (Chappell 1990). Fracture propagation in sedimentary rocks is influenced by the anisotropic strength and stiffness.

This plane of anisotropy is not present in granitic rocks and as such fracture propagation is related to the mineralogy, which is typically randomly distributed in granites. Studies of fracture propagation in granites show that fractures initiate as intragranular cracks and propagate parallel to the direction of maximum stress until a weakness causes a deviation in the propagation direction (Moore and Lockner 1995). These weaknesses have been observed in Westerly granite to occur within and along the edges of quartz crystals. However; induced fractures only follow the grain boundaries 25% of the time (Moore and Lockner 1995).

Fracture propagation in sedimentary rocks has been shown to be strongly dependent on the orientation of the propagating crack with respect to the bedding plane (Hoagland et al. 1973). Experimental fracture propagation studies conducted by Hoagland et al. (1973) showed that it requires substantially less energy to propagate a fracture along bedding rather than perpendicular to it. The propagating crack tries to take the path of least resistance and therefore will propagate toward bedding planes or other weaknesses in the rock mass.

The influence of bedding planes on the propagation path of a fracture has been observed in unconfined compression testing conducted on limestones for this thesis and on claystone tested
by Amann et al. (2011). The horizontal bedding in the UCS test causes the fracture to side step, as shown in Figure 7.1, since it requires less energy to propagate along a bedding plane. Amann et al. (2011) suggests that slight differences in stiffness in the sedimentary layers causes this side step pattern to develop. According to Amann et al. (2011) as the axial cracks form in a stiffer layer and propagate towards the less stiff layer, shear stresses develop at the interface causing a shear fracture to form which deviates the fracture from propagating in the axial direction.

Figure 7.1: Fracture pattern in unconfined compression tests on a) Lockport limestone from Niagara Falls, Canada (this study), and b) Opalinus Claystone from Mont Terri, Switzerland (from Amann et al. 2011) showing side step of failure surface due to near horizontal bedding plane influence on the propagation fracture.
At the grain scale level other influences, such contrasting stiffness in minerals or matrix – grain stiffness contrasts can cause deviation in fracture growth or arrest fracture growth. An example fracture, in the Salem limestone, is shown in Figure 7.2a below, where the fracture has passed through the matrix and around grains (intergranular). Some portions of the fracture pass through grains (intragranular), see top of Figure 7.2a. This behaviour is in contrast to the behaviour observed in granites, as discussed above, where fractures predominantly pass through grains. This is likely due to stiffness contrasts between the grains and matrix in sedimentary rocks. An example from the Opalinus claystone demonstrates this more clearly, as shown in Figure 7.2b. In this example the matrix is the dominant component of the rock mass and fossil fragments “float” in the matrix. Klinkenberg et al. (2009) report that samples with elongate fossil fragments (as shown in Figure 7.2b) are weaker, in compression, than samples with lower aspect grain ratios. This suggests that the grain boundaries represent weaknesses which can be exploited for propagation if the fossil is oriented parallel to sub-parallel to the propagation direction. Figure 7.2b also indicates that if the fossil is oriented with its long axis perpendicular to the propagation direction that it can in fact cause the crack to deviate, losing energy and in some cases arresting propagation. The micro-scale observations discussed above also apply at the excavation scale.

At the Mont Terri URL, west of the town of St Ursanne, Switzerland (Figure 7.3a), induced fracture characteristics have been studied in detail at the excavation scale as well. The Mont Terri facility is located off the security gallery of the A16 motorway tunnel, which runs under Mont Terri, part of the Jura Mountains (Figure 7.3b). The laboratory is situated within a limb of the anticline making up Mont Terri, which lies at the transition from the folded Jura (to the South) and the Tabular Jura (to the North), as shown in Figure 7.3b. The strata are part of a frontal thrust system consisting of sedimentary rocks of the Dogger and the detached Malm.
Figure 7.2: Crack(s) in (top) a Salem limestone sample (Hoagland et al. 1973) and (bottom) an Opalinus claystone sample (Klinkenberg et al. 2009), showing fractures in the matrix, along grain boundaries and to a lesser extent through grains.
Figure 7.3: Mont Terri URL, showing a) general location map within Switzerland and local area near St.Ursanne, b) geological cross section along the Mont Terri road tunnel, and c) a plan view of the URL with geology (modified from Corkum and Martin 2007, with updated gallery layout and geology from Maineult et al. 2013).
The Dogger units have been over thrust onto limestones of the Malm or Tabular Jura (Bossart & Thury 2008). The investigations at the URL are focused on the Opalinus claystone, which is a target formation in Switzerland for underground nuclear waste storage because of its low permeability, self-sealing potential and constructability.

Observations of the constructability within the Opalinus and at Mont Terri in particular were conducted during the construction of a reconnaissance gallery for the motorway tunnel in 1989 (Thury and Bossart 2007). Following from the promising observations during the road tunnel construction and interest from the international community to investigate argillaceous rocks for nuclear waste disposal underground, construction began in 1996 of the underground research facility (Thury and Bossart 2007). Investigations have the basic aim to:

1. understand the characteristics, processes and mechanisms in claystone,
2. understand repository-induced perturbations through experiments, and
3. to demonstrate waste repository performance during operation and post-closure.

Chappell (1990) indicates that anisotropy is stress dependent and therefore the orientation and magnitude play an important role in the development of the EDZs. The stresses at the Mont Terri laboratory have been measured by a variety of methods including; overcoring, borehole slotter, undercoring, and hydraulic fracturing. In addition to measurements specific to determining the stresses, other observations have been used, such as borehole breakouts.

One important finding from the stress measurements was that the galleries influence the surrounding rock mass up to two tunnel diameters away (the excavation influence zone) and that measurement techniques relying on elastic strain can be impeded by the sensitive nature of the Opalinus clay to moisture (Bossart and Thury 2008). The resulting stresses from the measurement
campaigns and from numerical analysis of the undercoring, as well as borehole breakout observations, by Martin and Lanyon (2003) are shown in Figure 7.4.

The strength, stiffness, and stress all impact the development of the EDZs. As previously mentioned, bedding plays an important role in the development of the EDZs, as well. At Mont Terri, it was observed that the EDZs were deeper when tunnelling parallel to the strike of bedding than when tunnelling perpendicular (Bossart and Thury 2008). This influence is observed at both the borehole and excavation scales.

At the borehole scale, observations by Labiouse and Vietor (2013) and Corkum (2006), for example, indicate that the interaction between the stress field and bedding controls the fracture orientation and the geometry of the borehole breakout.

![Figure 7.4](image.png)

**Figure 7.4**: Final stress magnitudes and orientation (lower hemisphere projection) at the Mont Terri Rock Laboratory. The dashed line represents bedding and tectonic shears are represented by solid lines. The arrows indicate the alignment of excavations (from Yong et al. 2010, after Martin and Lanyon 2003).
In Figure 7.5a and 7.5b, the breakout was induced by first loading the sample back up to
the in-situ stress conditions and moisture content and then unloading and draining. Figure 7.5a
shows the 600 mm diameter test sample and Figure 7.5b shows a zoom into the damage zone,
which developed only after the sample was removed from the testing cell. The main fractures are
parallel to bedding and the beams have buckled into the central borehole. The damage zone is
approximately as wide as the central borehole. Due to the anisotropic stiffness the rock around the
packer has compressed more in a direction perpendicular to the bedding orientation than parallel
to bedding. On unloading fractures propagate along bedding planes as the rock mass buckles into
the central hole and as the beams deflect into the hole they buckle at the mid span. This induced
failure using the packer shows a different failure geometry than that induced around boreholes
observed in-situ at the Mont Terri Laboratory. In-situ observations by Labiouse and Vietor (2013)
are shown in Figures 7.5c and d, which show a time dependent progression of borehole breakout.
First (Figure 7.5c) the rock mass wants to shear into the borehole, however beds which are fully
connected (a complete beam) limit this shearing until these beams begin to buckle / spall into the
borehole. This first stage results in crushing of the complete beam and shear along bedding. This
is followed by buckling three months later in the same borehole (Figure 7.5d), although at a
different location due to access issues for the borehole camera. Observations by Corkum (2006)
show that the geometry of the break out is closer to the notch shape common with the brittle
failure processes in granite (Martin 1997, Diederichs 2007). However; in the long term the
borehole geometry becomes rectangular due to progressive failure (Figure 7.5e). The square is
aligned with the bedding plane orientation and is similar to the geometry shown by Labiouse and
Vietor (2013). A close examination of the failed beams in Figure 7.5b, shows a zone of crushing
in the central region, which is similar to the process zone described by Martin (1993).
Figure 7.5: Borehole breakouts at the Mont Terri Underground Laboratory, showing a) the 600 mm overcore sample, b) zoom into the damage zone induced by unloading a packer, c) stress induced failure with shearing along bedding (black arrows) and buckling (white arrow), d) extensive buckling taken 3 months after (c), different location in same borehole, and e) long term borehole breakout geometry. Bedding plane orientation is indicated by the dashed line (compiled from Labiouse and Vietor 2013, Corkum 2006).
Recent work by Amann (per. communication) has shown that a notch indeed forms first and is followed by buckling as a delayed response. This failure mechanism is most severe when the borehole axis is parallel to the bedding plane. Labiouse and Vietor (2013) indicate that when the borehole axis is perpendicular to the bedding plane, isotropic convergence is observed and that a notch does not develop.

The observations at the Mont Terri Laboratory at the borehole scale indicate that the failure mechanism is associated with the orientation of the excavation with respect to the plane of anisotropy created by the bedding in the Opalinus claystone as well as the stress orientation.

At the excavation scale similar mechanisms, as discussed above, impact the development of fractures around the excavation. Bedding orientation was observed to influence the failure geometry at Mont Terri (Blumling et al. 2007) in a similar manner to that observed by the author at the Niagara Tunnel Project (Chapter 5). Observations by Yong et al. (2010) at the intersection of the main Gallery 04 and a side niche called the EZ-B (Figure 7.3c) showed that three sets of induced fractures (IF) developed due to the excavation. IF1 is oriented sub-parallel with the axis of Gallery 04 and is the most frequent. IF2 is oriented oblique to the axis of Gallery 04 and IF3 is sub-parallel to bedding. Yong et al. (2010) reports that the IF2 fractures occur up to 0.5 m from the intersection and the IF1 fractures up to 1 m. See the plan view of Figure 7.3.

As shown in Figure 7.6 the induced fracture orientations are different on the east wall than on the west wall. On the east wall the fractures are nearly parallel with the gallery and on the west wall they are perpendicular to bedding. Yong et al. (2010) contributes this difference to the tectonic shears and the kinematic freedom with respect to their three-dimensional orientation.
Figure 7.6: Induced fracture (IF) mapping at the intersection of Gallery 04 and EZ-B Niche and their interaction with tectonic shears (F#) (after Yong et al. 2010).
In Figure 7.6 the SSE shears dip parallel to the axis of the EZ-B niche and their strike is roughly 15° off the axis of Gallery 04. This means that not all the same tectonic shears daylight into Gallery 04 on the east and west sides. Despite this difference, the induced fractures tend to terminate at the tectonic shears. The induced fracture frequency decreases away from the excavation surface in both mapped sections.

A study by Bossart et al. (2002) mapped the fracture frequency at various locations around the Mont Terri Laboratory, with a particular focus on determining the difference between excavation methods. A summary of these results are shown in Figure 7.7. The measurements are based on pneumatic and hydraulic testing from within boreholes, as well as fracture frequency from over coring. Based on these findings, Bossart et al. (2002) also present a conceptual model of the excavation damage zone, which includes the influence of bedding orientation with respect to the excavation orientation and unloading direction, as shown in Figure 7.8.

Extensive research at Mont Terri, as discussed above, and at other underground research laboratories (Armand et al. 2013, Blumling et al. 2007) has demonstrated the importance of anisotropy, caused by bedding, on the development of the excavation damage zone.

Work by Perras (2009) demonstrated numerically that the stress flow around an underground excavation in an anisotropic rock mass is different than the equivalent isotropic rock mass model. Numerical work by Lisjak et al. (2013) using the combined finite-discrete element method (FEM/DEM) in Y-Code showed that fractures propagate parallel to bedding or that they propagate perpendicular to bedding until they deviate parallel. Numerical modeling has been used to understand the development of the EDZs in more detail and to develop strategies for delineating the dimensions of the EDZs.
Figure 7.7: Summary of EDZ measurements at the Mont Terri Laboratory (after Bossart et al. 2002), where a represents the dimension of the inner EDZ and the subscripts h and v stand for horizontal and vertical, respectively. The out EDZ (b) dimensions have not been reported specifically.

<table>
<thead>
<tr>
<th>Excavation method (locations indicated in Figure 3)</th>
<th>Extent of EDZ [m] (see drawing)</th>
<th>Frequency of EDZ fractures [m⁻¹]</th>
<th>Orientation of EDZ fractures</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Pneumatic hammering</strong> data from only 2 boreholes in new gallery</td>
<td>$a_h \approx 0.70$</td>
<td>$\approx 19$</td>
<td>Subvertical dip, parallel to tunnel wall, oblique to bedding</td>
</tr>
<tr>
<td></td>
<td>$a_v &gt; 1.2$</td>
<td>not known</td>
<td>Inclined, subparallel to bedding</td>
</tr>
<tr>
<td><strong>Blasting</strong> data from 10 boreholes in security and new gallery</td>
<td>$0.60 \leq a_h \leq 1.10$</td>
<td>$1 - 7$ (along $a_h$)</td>
<td>Subvertical dip, parallel to tunnel wall, oblique to bedding</td>
</tr>
<tr>
<td></td>
<td>$0.30 \leq a_v \leq 1.25$</td>
<td>$4 - 15$ (along $a_v$)</td>
<td>Inclined, subparallel to bedding</td>
</tr>
<tr>
<td>data from 8 side niches in security tunnel</td>
<td>$0.65 \leq a_h \leq 0.75$</td>
<td>$4 - 32$ (along $a_h$)</td>
<td>Subvertical dip, parallel to tunnel wall, oblique to bedding</td>
</tr>
<tr>
<td></td>
<td>$0.35 \leq b_h \leq 1.20$</td>
<td>$0 - 12$ (along $b_h$)</td>
<td></td>
</tr>
<tr>
<td><strong>Road header</strong> data from 8 boreholes in new gallery</td>
<td>$0.10 \leq a_h \leq 0.60$</td>
<td>$1 - 9$ (along $a_h$)</td>
<td>Subvertical dip, parallel to tunnel wall, oblique to bedding</td>
</tr>
<tr>
<td></td>
<td>$0.30 \leq a_v \leq 0.90$</td>
<td>$2 - 8$ (along $a_v$)</td>
<td>Inclined, subparallel to bedding</td>
</tr>
</tbody>
</table>
7.2 Numerical Rock Mass Response

The excavation damage zone delineation in numerical models is sensitive to the choice of constitutive model. The response of the numerical rock mass is controlled by the constitutive behaviour, which generally can be of a shearing, strain-weakening/hardening, or brittle nature.
For rocks typically considered for the storage of nuclear waste underground the in-situ failure process, if it occurs, is typically of a brittle nature. The exception is of course repositories being planned in salt deposits. For completeness shear based behaviour, as it applies to hard rock, is discussed briefly below. The numerical behaviour of salt around underground excavations is beyond the scope of this thesis.

The onset of damage at the excavation wall is controlled by the induced wall stresses and the effective unconfined compressive strength. This effective UCS is a percentage of the laboratory maximum UCS. For rock masses with structural features such as joints and foliation, this value is reduced further according to methodologies such as the GSI (Hoek et al. 2002) and Hoek-Brown (1997) system.

A preliminary analysis can be conducted using a Hoek-Brown (1997) failure envelope and a perfectly plastic constitutive behaviour, which is a shear based behavioural model. This can be useful to estimate the extent of plastic yielding, which marks the transition from the EIZ to the EDZ, as shown in Figure 7.9. It should be noted that it is not possible to definitively determine the dimensions of the HDZ, since the constitutive behaviour does not require a stress drop post yield. Also the zone of uncertainty in Figure 7.9 is used to convey that the transition is gradual, similarly in Figure 7.10.

Without a stress drop post yield there is a gradual reduction in the stress moving toward the excavation wall, as shown in Figure 7.9 according to the minimum stress. By including a residual failure envelop, such that the rock is weaker post yield, a less gradual transition of the minimum stress from the outer edge of the plastic yield zone to the excavation wall occurs (Figure 7.10).
Strain-weakening models simulate a reduction in strength within the yield zone and can provide a more realistic assessment of the EDZ (yield) limits. In addition, it is possible to identify the extent of the HDZ, although this is still not the optimum tool for analyzing this zone. The HDZ contains open and likely connected fractures. Conventional continuum codes do not adequately address the discontinuum mechanics which develop in the HDZ.

Figure 7.9: Perfectly plastic numerical rock mass response, showing behaviour with respect to the distance away from the excavation wall measured along a line through the furthest plastic yield point. A circular excavation of radius 5 m, an in-plane stress ratio of 1.2 and at 600 m depth modelled in Phase2, by Rocscience (modified from Perras et al. 2010).
Figure 7.10: Strain weakening numerical rock mass response, showing behaviour with respect to the distance away from the excavation wall measured along a line through the furthest plastic yield point. A circular excavation of radius 5 m, an in-plane stress ratio of 1.2 and at 600 m deep modelled in Phase2, by Rocscience (from Perras et al. 2010).

Nevertheless, continuum based approaches can still be used to approximate the potential for a significant HDZ. Figure 7.10 illustrates a subjective approach to HDZ delineation in continuum models by using the point of minimum stress increase, which coincides with rapidly
decreasing shear volumetric strains (moving away from the excavation boundary). In this approach the HDZ will be conservatively over-estimated.

7.2.1 Brittle Behaviour

For brittle rocks that have a dominant tendency to split or spall near the excavation boundary, the conceptual model of Diederichs (2007), shown in Figure 7.11, can be adopted to delineate the three damage zones around the excavation. This conceptual model is called the Damage Initiation and Spalling Limit (DISL). Spalling is a process in which extensile fracture growth dominates over shear crack and shear zone generation.

In addition, while massive (high GSI) rocks may be spall prone, the accumulation of joint density and number of joint sets, as well as decreased joint strength at lower GSI’s will result in a transition from spalling to inter-block shear failure characteristic of a composite rock mass.

Diederichs (2007) suggested a method to determine when GSI or DISL modeling approaches should be conducted (see Table 7.1). The limits are determined by using the rock mass GSI value and the strength ratio (UCS/T).

The transition between spalling in massive rock and distributed shearing in jointed or damaged rock can clearly be seen in Figure 7.12. Here the intact portion of rock is assigned a spalling criterion. The properties of the intact rock are unchanged from model to model but the number of discrete joint planes increases while the quality of the joint planes (shear strength) decreases from the first to the last model.

The equivalent GSI is shown for each model in Figure 7.12. In the first model spalling is dominant while by GSI 55, spall fracture within the intact blocks vanishes with joint shear taking over as the dominant yield mechanism. This supports the model selection process outlined in Table 7.1.
Figure 7.11: Conceptual model for rock damage after Diederichs (2003). CI and CD are lab-based stress thresholds for Crack Initiation (CI) and Crack Damage (CD). Behavioural domains include (A) Spalling, (B) Transitional and (C) Shear Fracture as indicated for the different areas of the conceptual model, with in-situ examples from Diederichs (2007).
Table 7.1: Model selection based on strength ratio (Unconfined Compressive Strength, UCS, divided by tensile strength, T) and rock mass quality (GSI). DISL = spalling approach, GSI = rock strength based on GSI and Hoek-Brown strength criteria (after Diederichs 2007). The order of the methods indicates the most appropriate.

<table>
<thead>
<tr>
<th>Strength Ratio UCS/T</th>
<th>GSI &lt; 55</th>
<th>55 &lt; GSI &lt; 65</th>
<th>65 &lt; GSI &lt; 80</th>
<th>GSI &gt; 80</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;8</td>
<td>GSI</td>
<td>GSI</td>
<td>GSI</td>
<td>GSI</td>
</tr>
<tr>
<td>9 to 15</td>
<td>GSI</td>
<td>GSI</td>
<td>GSI</td>
<td>GSI/DISL</td>
</tr>
<tr>
<td>15 to 20</td>
<td>GSI</td>
<td>GSI/DISL</td>
<td>DISL/GSI</td>
<td>DISL</td>
</tr>
<tr>
<td>&gt;20</td>
<td>GSI</td>
<td>GSI/DISL</td>
<td>DISL</td>
<td>DISL</td>
</tr>
</tbody>
</table>

Figure 7.12: Transition from spalling failure through intact rock at GSI=90 to failure controlled entirely by joint slip and block movement at GSI=45. The intact rock in this case has a UCS of 100MPa and a m_i of 20. The transition between spall dominated and rock mass (GSI) dominated behaviour occurs at approximately GSI=65 in this case. This is consistent with the observational limits suggested by Diederichs (2007). Displacements are contoured and the scale is the same for all models (after Perras and Diederichs 2010).
The damage zones have been mapped to the conceptual failure envelopes of Diederichs (2007) in Figure 7.13. The EDZ is defined in this model as the onset of damage represented by exceedance of the damage initiation limit, which is defined by CI. There is a large difference, however between the strains accumulated in the inner EDZ (low confinement) and the outer EDZ (high confinement). A zone inside the EDZ boundary which has dramatically elevated shear strain and low confinement represents the HDZ in this model. The concepts defining the EDZs for the DISL approach are also applicable for anisotropic behaviour.

7.2.2 Anisotropic Behaviour

Chappel (1990) found that anisotropy is both modulus and stress dependent and therefore not all rock masses with parallel structure or fabric will exhibit the same degree of anisotropic behaviour. Similar rock masses under different stress regimes will not necessarily behave in a similar manner.

Anisotropic stiffness can be induced in a continuum model by utilizing a joint network to create laminations (using joint elements) and rock beams (the intact rock between the joints), following the methods defined by Perras (2009). To conserve computational time, the laminated area does not need to extend to the full model domain, although it should extend well beyond the expected yield zone. To ensure compatibility between the laminated area and the non-laminated area the relationship for transversely isotropic elasticity can be used to scale the modulus, accounting for the joint normal stiffness ($K_N$) and spacing of the laminations ($T$). The moduli are balanced using Equation 7.1 (Brady and Brown 2006).

$$\frac{1}{E_{rm}} = \frac{1}{E_{beam}} + \frac{1}{K_N T}$$ (7.1)
To validate the anisotropic modeling approach, Perras (2009) compared an isotropic (no laminations) elastic model with an anisotropic model, where the rock and laminations were
elastic, and a transversely isotropic elastic model. The results are shown in Figure 7.14, which indicate that the stress contours around the excavation are different for the isotropic model than the other two models. The transversely isotropic elastic model gives the same results as the anisotropic model when both the rock and laminations are elastic.

Transversely isotropic plastic constitutive models are only now being implemented in beta software packages for commercial use. The anisotropic plasticity method (Perras 2009) is a suitable work around, which will be shown later in Chapter 9 to capture both realistic yield zones and deformation behaviour. This follows from the work of Perras (2009) who studied the influence of lamination thickness on the failure mechanism in numerical models, utilizing vertical crown deflections to compare models.

Figure 7.14: Stress contour plot around a 16m diameter tunnel for an isotropic elastic model, an anisotropic elastic model (horizontal joint elements), and a transversely isotropic elastic model. Each model is representative of a lamination thickness of 280 mm at 150 m deep with a $K_o$ ratio of 3, modelled in Phase2, by Rocscience (modified from Perras 2009).
Figure 7.15: Comparison of modeling methods at various lamination thicknesses using vertical crown deflection, showing four different behaviours; 1) Plastic yield self-limiting, 2) Multi-beam coupling, 3) Stress channeling and 4) Isotropic (after Perras 2009).

Four major mechanisms were defined; 1) Plastic yield self-limiting, 2) Multi-beam coupling, 3) Stress channeling and 4) Isotropic; and their relationship to the normalized lamination thickness is shown in Figure 7.15. The first mechanism, plastic yield self-limiting, shows that the extent of plastic yield begins to stabilize in the finite element models and reducing the lamination thickness only has an elastic effect on the crown deflections (Perras 2009). Multi-beam coupling is analogous to the voussoir model, first proposed by Evans (1941), although as shown by Perras (2009) the circular geometry and stress flow through the beams allows the system to deform more without complete snap through as would be predicted using the iterative
approach of Diederichs and Kaiser (1999). As the thickness decreases, stresses begin to channel through one or two beams and in effect provide a measure of confinement which stiffens the beams. Ultimately the lamination thickness becomes large enough that isotropic modeling gives similar results to the anisotropic method.

The relationship between normalized crown deflection and lamination thickness can be used, in conjunction with the laboratory testing and excavation observations of brittle failure discussed in Chapters 3, 4 and 5, to describe the brittle failure process in sedimentary rocks.

**7.3 Conceptual Development of EDZs in Sedimentary Rocks**

Based on the research discussed thus far in the thesis, a conceptual model is presented to describe the development of EDZs in sedimentary rocks where the bedding and maximum stress are both horizontal. This is the typical setting in sedimentary basins which have not been or only weakly disturbed by tectonic activity (i.e. no folding), which is the ideal sedimentary environment for nuclear waste disposal underground.

The process of EDZ development has been broken down into 4 main stages, as outlined in Figure 7.16. In Stage 1 the damage initiation begins developing in the area of maximum tangential stress along the excavation boundary and microcracks form parallel to bedding.

In Stage 2 the bedding planes become actively involved in the failure process. Brittle fractures begin to propagate towards and along bedding planes, as these planes are the direction of least resistance. The shearing and splitting along the bedding planes creates beams of intact rock which deflect into the excavation and begin to load the rock in the haunch area.

In Stage 3 the haunch area becomes over loaded and fractures begin to develop; first parallel to the excavation boundary and then rotating to sub-vertical moving away from the excavation. The fracture orientation changes due to the stress flow around the excavation.
Figure 7.16: Conceptual stages of damage development in horizontally laminated sedimentary rock (massive) with the maximum principle stress oriented horizontally.
The stress flow begins similar to that of homogenous rocks, but deviates due to the anisotropic nature of the sedimentary rock and the fracture propagation direction preference towards bedding planes. The loading of the haunch area from the deflecting beams, the fracturing and the distressing of the haunch area causes this rock to fall out.

In Stage 4, with the haunch rock removed the rock mass above the tunnel can now behave in a similar fashion to a voussoir beam (Diederichs and Kaiser 1999). Observations by the author from the Niagara Tunnel Project suggest in mudrocks that mid-span snap through and crushing are the main failure mechanisms occurring in Stage 4.

The width of the notch is a function of the stiffness of the rock mass and magnitude of the tangential stress at the notch tip. A numerical study by Perras (2009) indicated that at lower stress to stiffness ratios the depth of the notch is also lower, to a point. There is of course a transition point where purely gravity driven instability takes over from the stress assisted failure. At low lamination thickness to tunnel radius ratios this transition occurs at a stress to stiffness ratio of around 0.001.

Observations at the Niagara Tunnel Project (Chapter 5) and at Meuse/Haute-Marne URL suggest that the out of plane fractures (Figure 7.16) are orientated such that they form a chevron with the narrow area orientated towards the face and the maximum horizontal stress direction (Armand et al. 2013).

Rock support can help to stabilize the notch and stop excessive over break from occurring due to gravity raveling, post Stage 4. The rock support will not stop crack initiation from occurring, however, if installed close to the excavation face it can minimize crack propagation and rock mass dilation. By reducing the rock mass dilation the post Stage 4 depth of damage can be reduced. The orientation of the bedding planes also plays a role in the failure mechanism.
Figure 7.17: Conceptual EDZ limits and fracture patterns in horizontally bedded sedimentary rock with different stress rotations relative to bedding.
Applying the conceptual failure process in combination with the observations presented in this research and the work of Wermeille and Bossart (1999), the generalized expected EDZ geometries and character of the fracture network are shown in Figure 7.17.

A variety of different stress orientations, relative to the horizontal bedding, are used to illustrate the change in the EDZ due to the influence of the bedding-stress interaction. As maximum principal stress rotates from parallel to bedding to perpendicular, there is more confinement applied to the bedding surfaces. This increases the shear resistance on the bedding and therefore reduces the depth of damage.

In sedimentary rocks which are sensitive to changes in moisture or deteriorate over time, the EDZ character will also change if deformation is allowed to occur. This could potentially expand the EDZ and will certainly expand the HDZ. Very little confining pressure is likely needed to stabilize the excavation surface from time dependent deterioration, as long as the rock mass is not susceptible to large creep strains.

### 7.4 Conclusions
An understanding of how the EDZ develops in sedimentary rocks has been established, based on laboratory testing focused on crack propagation and field observations focused on damage mechanisms and fracture orientations. The bedding or other linear features (calcite veins) influence the crack propagation at the laboratory and field scales. The bedding also creates anisotropic stiffness of the rock mass, which controls the behaviour of the rock beams. At low rock mass stiffnesses, the deformation mechanisms are influenced more so than at higher stiffnesses. Based on the kinematics, the stress field and the weakness plane of the bedding fractures tend to form parallel or perpendicular to the bedding. When the stress field is oriented such that there is increased normal stress on the bedding, the dimensions of the EDZ will be less
compared with when the maximum stress is oriented parallel to bedding. This interaction between
the stress field and the bedding controls the geometry of the EDZ. Based on research in the
Opalinus clayshale when Ko is close to 1 the EDZ will tend to form into a square geometry and
as the Ko ratio increases the classic stress notch will form. With an understanding of how the
EDZ develops and the characteristics of the fracture network, predictive numerical indicators
have been evaluated in Part II of this thesis.

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PART III: Predicting the Dimensions of the Excavation Damage Zones
Chapter 8: Predicting the dimensions of Excavation Damage Zones using continuum mechanics

8.1 Abstract
During the construction process of an underground excavation, the rock mass surrounding the tunnel, shaft, or cavern is damaged. In the case of blocky rock masses, this damage can be in the form of discontinuity shearing or dilation. For massive or moderately jointed rock masses at depth, this damage is typically in the form of new micro- to macro-fractures. Beyond this irreversibly damaged area, the stresses are modified. In modern nomenclature, these zones are collectively known as the excavation damage zones (EDZs). In the case of massive rock, the EDZ dimensions can be determined numerically using the appropriate laboratory test results to describe the rock, including the compressive and tensile strengths. With a large data set, it is possible to test the mean as well as the bounding input values to evaluate the results (for EDZ prediction) using statistical or probabilistic methods.

Using three data sets, a granite, a limestone, and a shale, as well as two nominal stress regimes per rock type, the variability in the input properties on the numerical dimensions of the EDZs was explored. Although discontinuum methods are gaining ground for damage analysis, it is still reasonable in many cases to use simpler continuum analysis methods for EDZ prediction. Both inner and outer excavation damage zones (EDZi and EDZo) could be differentiated using the reversal point in the volumetric strain (contraction to extension). This finding indicates the transition between a confined micro-damaged state (EDZo) and a potentially dilated EDZi.

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6 This chapter appears as submitted to an international journal with the following citation: Perras MA, Diederichs MS (2014) Predicting the dimensions of excavation damage zones using continuum mechanics. International Journal of Rock Mechanics and Mining Sciences.
outer boundary of the highly damaged zone, HDZ, is related to volumetric strain and a reduction in minor stress confinement. Guidelines are suggested for determining the EDZ dimensions around circular excavations using the Damage Initiation and Spalling Limit continuum approach. Sensitivity of the model results to key model inputs were evaluated; it was found that the tensile strength used produces the greatest variation in the EDZ dimensions.

The results were evaluated statistically to determine the best fit relation between the model inputs and the dimensions of the EDZs. Cumulative distributions for the EDZ dimensions were also determined. The highly damaged zone showed the least variability, whereas the EDZo showed the most. This method can be used to determine the depth of a cut-off structure to limit the flow through the damage zone, for example, at the required confidence level.

8.2 Introduction

The design of an underground excavation requires site-specific data about the subsurface. The level of complexity or the design life span will determine the volume of data necessary to adequately design the excavation. Extra costs associated with large numbers of standard tests or conducting non-standard tests can accumulate quickly, so one must understand what information is required for design, how it will be used in numerical simulations, and how it will influence the numerical results. A full sensitivity analysis of the parameters used in a numerical model for a typical engineering project is often too expensive. In this paper, the sensitivity of some common numerical input properties are explored, with respect to the resulting depth of yielding around a circular excavation for three rock types: granite, limestone, and shale.

8.2.1 Excavation Damage Zones

The excavation process damages the rock mass, producing zones of damage. This concept has been studied since the early 1980s in relation to nuclear waste disposal because it changes the
permeability due to the damage (Kelsall et al. 1984). Determining the depth of damage is important and is required to design cut-off structures to reduce flow along the damage zone. The terminology related to damage zones has changed because of the improved understanding of how the damage is induced and how it changes the permeability around the excavations. The terminology used herein is only slightly modified from past literature (e.g. Tsang et al. 2005) to make a clear distinction between where the rock mass is damaged and where elastic changes are occurring. The damage zones are traditionally referred to collectively as the excavation damage zones (EDZs) and the various zones are depicted in Figure 8.1. The density of excavation induced fractures decreases moving away from the excavation surface. The inner most zone is termed the highly damaged zone (HDZ). It comprises macroscopic (often visible) interconnected fractures. Moving outwards, the inner EDZ (EDZi) makes a gradual transition to the outer EDZ (EDZo). These zones contain micro-damaged rock with (inner) and without (outer) significant dilation.

Figure 8.1: The Excavation Damage Zones (HDZ, EDZ, EIZ) and the Construction Damage Zone. Note that the EIZ was referred to as the Excavation Disturbed Zone (EdZ) by Tsang et al. (2005) and was re-named due to potential confusion with the lower case “d” and the upper case “D” of the EdZ and EDZ, respectively.
Beyond the EDZs is a stress and/or strain influence zone that involves only elastic change, the Excavation Influence Zone or EIZ, called the Excavation disturbed Zone (EdZ) by Tsang et al. (2005); however, the authors of this work feel that the lowercase ‘d’ is too easily confused with the uppercase ‘D.’ In nature, the transition between these zones is gradational, and distinguishing between them can be difficult without in situ measurements to validate the numerical results.

Another important distinction is between the HDZ and the Construction Damage Zone (CDZ). Construction damage is purely a result of construction methods (blasting, for example). This form of damage can be reduced or nearly eliminated through precision rotary boring or tunnel boring machine excavation. In contrast, the HDZ can be purely the result of geometry and induced stress change (independent of excavation method) and in some cases can never be eliminated or significantly reduced. Close to the excavation surface an increase in porosity and permeability is caused by interconnected fractures within both the CDZ and the HDZ. The depth of damage is a function of the excavation method, stress regime, and rock mass properties. The degree of fracturing can cause the stresses to be shed further away from the excavation surface if it is severe enough. If the degree of fracturing in the HDZ is severe enough to cause post-peak weakening or softening and associated stress shedding, it is within the EDZi that the stresses are able to concentrate as the induced fractures become less inter-connected and the rock mass begins carrying the stress load. This zone gradually transitions into the EDZo, where fractures begin to occur under enough confinement such that fracture propagation is inhibited. Damage within the EDZo impacts the bulk hydraulic properties within this zone to a much smaller degree than in the zones closer to the excavation.
The outer limit of the EIZ is typically of minimal interest for a single excavation, as it occurs at a large distance from the excavation surface. The interaction of EIZ (and EDZ) with adjacent excavations are important and should be considered. Numerical methods can be used to capture the potential variations of the EDZ dimensions using the distributions of certain laboratory parameters as inputs.

EDZs have become an important consideration for designing underground structures which are intended to store a commodity with potentially mobile constituents such as gas, oil, or compressed air. It is also critical in cases where the surrounding rock is part of a barrier system to isolate wastes from human exposure, such as nuclear waste. EDZ prediction and evolution modelling is essential for designing bulkheads and cut-off structures which can inhibit the flow parallel to the excavation through the EDZs. Understanding and being able to design the dimensions and possible variation of the EDZs is crucial for the success of the project. The process of selecting appropriate input values, modelling, and determining the dimensions of the EDZs, the range of variability, and the sensitivity of the numerical results to the laboratory inputs are explored in this paper.

8.2.2 Basis for the Continuum Based Approach

The basis of this study was to develop an engineering design solution which could be implemented with standard engineering software to predict the dimensions of the EDZs. Numerically there are three main options; continuum, discontinuum and hybrid methods. Continuum based methods are easily implemented in standard design software, such as Phase2 from Rocscience which was used in this study. A continuum approach is applicable when the rock mass behaves in a unified manner and the influence of structure (joints, faults, etc.) is minimal or isolated with respect to the overall excavation behaviour. Nuclear waste repositories
will generally be constructed in massive rocks with only local structural influences. This make
continuum methods particularly well suited for determining the dimensions of the EDZo, since
the EDZo is a transition zone between no damage to micro cracking. It will be shown that within
the EDZi the volumetric strain is extensile and therefore micro cracks can propagate and become
interconnected.

Discontinuum and hybrid methods are better suited for understanding the transition from
micro to macro cracking with in the HDZ. However, since a cut-off should be designed to cross
the zone of interconnected damage, the continuum based approach examined in this paper is well
suited to determine the dimension of the transition from interconnected to isolated damage. The
predictions of the HDZ from continuum based approaches should be conservatively larger than
those predicted by discontinuum or hybrid approaches, although this has not been studies as part
of this research.

8.3 Numerical Model Inputs
A typical laboratory testing program includes uniaxial compressive strength (UCS) and tensile
tests. Triaxial compression tests are recommended by Hoek and Brown (1997) to determine the
intact rock strength ($\sigma_i$) and the intact rock material constant $m_i$. Triaxial test results can also be
used to determine the friction angle and cohesion if the Coulomb (typically referred to as Mohr-
Coulomb) criterion is to be used.

Other characteristics of the rock mass, such as joints, are often included by reducing the
intact rock properties to represent the rock mass; for example, the use of the Geological Strength
Index (GSI) by Hoek et al. (2002). The joints can be included explicitly and would require
estimates of the properties, such as normal and shear stiffness or roughness, and additional
measurements in situ or in the laboratory to determine the appropriate values.
Both the Hoek-Brown and Mohr-Coulomb criteria can include a dilation parameter for the rock mass. Dilation is a measure of the post-yield behaviour of a rock which is confinement and plastic-strain dependent. For Mohr-Coulomb, a rule of thumb used by some practitioners is the use of a constant dilation angle equal to or less than half the friction angle. Zhao and Cai (2010) demonstrated that a constant dilation angle does not simulate the non-linear deformation behaviour near the excavation boundary because the dilation angle is confinement and plastic-strain dependent. Determining a non-constant dilation angle using current methods requires the use of many input parameters and can only be implemented in modelling software which allows parameters to change with the plastic shear strain increment. Dilation has not been considered in this part of the study on the prediction of the EDZ dimensions.

8.3.1 Intact Properties
The typical intact properties discussed above are required to determine numerical model input parameters when using the Hoek-Brown failure criteria (Hoek et al. 2002). Coupled with GSI, the rock mass properties for input into numerical models can be determined. This approach is called the GSI system in this paper. This rock mass strength approach is applicable for rock masses with a GSI < 75.

For rock masses with GSI > 75, Diederichs (2007) developed a method to represent brittle spalling behaviour with the generalized Hoek-Brown peak and residual parameters, as these are standard inputs for engineering design software. This method is called the Damage Initiation and Spalling Limit (DISL), which captures the confinement dependency of the brittle spalling process. This dependency means that as confinement increases, away from the excavation surface, plastic yielding is suppressed numerically. The method requires the Crack Initiation (CI) and UCS thresholds, and tensile strength as input properties from laboratory testing.
The CI threshold and Crack Propagation (CD) can be determined using strain measurements or acoustic emissions (AE). Several strain methods are available in the literature (Martin 1993, Eberhardt et al. 1998, Ghazvinian et al. 2012a) and include using the lateral, volumetric, and crack volumetric strains or calculating the inverse tangent lateral stiffness or the tangent modulus (Ghazvinian 2010). These methods predict the strength thresholds at changes in the stress-strain curve slope. The deviation of lateral strains, measured at the midpoint of a UCS or triaxial sample, from linearity indicates a switch from crack closure to crack initiation within the sample (Brace et al. 1966, Bieniawski 1967, Lajtai and Lajtai 1974). The cracks initially form parallel to the maximum stress on the sample, typically the loading direction; therefore, the lateral strain rate increases once the cracks begin to form at CI. The deviation from linearity of the lateral strain can be difficult to detect in some cases. Diederichs and Martin (2010) presented the crack volumetric strain reversal point as a potential indicator for CI because it corresponds to the onset of dilation in the sample. The crack volumetric strain is calculated by subtracting the elastic volumetric strain from the volumetric strain. The elastic volumetric strain is a function of the modulus and Poisson’s ratio, which change throughout the test. The changing values make this method of selecting CI sensitive to the range over which the modulus and Poisson’s ratio are determined. Eberhardt et al. (1998) discuss the sensitivity of this method on the reported CI values.

The volumetric strain reversal point can be used empirically to indicate the CD threshold. Once the sample reaches this threshold, failure occurs quickly, relative to the full duration of the test. This threshold also corresponds to rapid increases in the lateral and axial strains of the sample; again, however, the exact point is difficult to tell from the measured strains of the sample (lateral and axial), as typically the stress-volumetric strain curve has a very shallow curvature at
the reversal point. Inverse tangent lateral stiffness and tangent modulus methods of determining CI and CD, respectively, have more recently been proposed (Ghazvinian 2010). The inverse tangent lateral stiffness method, proposed by Ghazvinian (2010), is a moving-point regression analysis of the lateral strain versus axial stress plot. CI is identified as the inflection point of the inverse tangent lateral stiffness versus axial stress plot. Similarly, the tangent modulus method utilises the change in slope to determine CD. The tangent modulus is defined as the moving-point regression of the axial stress versus axial strain plot. When the tangent modulus is plotted against the axial stress, changes in the slope are amplified, and CD can be picked at the point where the tangent modulus deviates from linearity. This method is also useful for determining the crack closure threshold for samples which have been damaged prior to testing (Ghazvinian 2010).

Similarly, Scholz (1968) found that changes in the AE event rates during testing correlated closely with stress-strain-based indicators for damage threshold detection. Using changes in the slope of the AE event rate versus stress level has also been shown to give similar damage threshold levels to strain-based methods for non-granitoid rocks (e.g. Amann et al. 2011, Ghazvinian et al. 2012b, Perras et al. 2012).

It has been proposed (Eberhardt et al. 1998, Diederichs et al. 2004, Ghazvinian et al. 2012a) that the damage thresholds detected by AE methods can be used to determine lower and upper bound values, particularly for CI, as the emissions are detected from within the sample. Strains, however, are detected at the surface of the sample and therefore a delayed response to the loading conditions can occur at the surface of the sample, causing the strain based CI and CD values to be slightly higher than the lower bound AE values (Ghazvinian et al. 2012a).

Diederichs (1999, 2003) indicated that the CI threshold is a robust indicator of the long-term strength of the rock mass and that the crack damage (CD) threshold is representative of the
short-term strength. These critical strength thresholds, as well as UCS for a series of tests on a granite, limestone, and shale, are used to determine the variation in the numerical input properties.

8.3.2 Laboratory Results for Numerical Input

Forsmark granite samples tested as part of the International Society of Rock Mechanics (ISRM) Spalling Commission inter-laboratory testing program (Ghazvinian et al. 2012a) and Cobourg limestone and Queenston shale samples tested for the Low and Intermediate Level Nuclear Waste Deep Geological Repository (Gorski et al. 2009, 2010, 2011) were used to explore the impact of variations in the input rock properties on numerically predicted dimensions of the EDZs. A summary of the laboratory testing results is discussed briefly for completeness.

The granite, limestone, and shale samples were tested in uniaxial compression with LVDTs and strain gauges to measure the strains. The CI and CD thresholds were determined by the laboratories as the axial stress at the reversal point in the crack volumetric strain ($\varepsilon_{cv}$) and the reversal point of volumetric strain ($\varepsilon_{vol}$), respectively, as suggested by Diederichs and Martin (2010). The reported values, in the documents outlined previously, were used directly. The strain-based CI values were taken as an upper bound threshold, as discussed by Ghazvinian et al. (2012a). Lower and upper bound AE-derived CI values were determined by Ghazvinian et al. (2012b) for the limestone samples only. The reported values are included in Appendix A.

The distributions of CI (upper bound), CD, and UCS values for the granite, limestone, and shale are shown in Figure 8.2. The ranges of values are similar for all the damage thresholds of the granite. The range of CI values for the limestone is smaller than the range for CD and UCS. The range of CD values is comparable to the range in UCS, and their distributions are similar for the limestone.
Figure 8.2: Crack initiation, crack damage, and compressive strength distributions for the a) granite (from Ghazvinian et al. 2012a), b) limestone (Gorski et al. 2009-2011), and c) shale (Gorski et al. 2009-2011) data sets with inset photographs showing the rock character.
This finding could be contributed to the high quality of sampling and testing conducted with these samples or perhaps a characteristic of the limestone itself, as similar trends for the granite data set are absent. This absence could be partly caused by the mineralogical character of the rocks, as the granite is mostly composed of minerals with the same stiffness and strength. The limestone contains argillaceous bands and wisps, the dark portions of the inset photo in Figure 8.2b, which can suppress crack propagation due to contrasting stiffness. This stiffness contrast forces new cracks to initiate elsewhere until a critical threshold is reached where cracks can propagate through the argillaceous band. Once this critical state is reached, the cracks propagate quickly and the sample reaches peak strength shortly after the CD threshold is reached.

The shale has a much narrower range of values, compared with the granite and limestone. There are distinct peaks for the damage thresholds; however, there is some overlap in the distributions. The range for CI is smaller than that of CD and UCS for the shale, similar to the limestone. Although less obvious in the inset photo of the Queenston in Figure 8.2c, variations of silt- and clay-rich layers within the Queenston samples influence the damage thresholds.

Laboratory tests demonstrate that with higher silt content CD and UCS thresholds are higher, as expected, but that CI is largely unaffected for samples from the Niagara region (Ghazvinian et al. 2013). The siltstone content in the Queenston samples from the Deep Geological Repository site in Canada were not measured; however, silt content could be an influencing factor in a similar manner as the argillaceous bands in the Cobourg limestone.

Tensile strength is often measured indirectly and most commonly by the Brazilian Tensile Strength (BTS) method. Because typically the BTS test is conducted on different samples (spatially) from the UCS samples, it is difficult to correlate the tensile strength with the crack
damage thresholds. Depth can be used, however, and for the data sets used in this paper limited test results correlate with the same depths at which the UCS samples were tested. Perras and Diederichs (2014) showed that the tensile strength can be estimated from the CI threshold.

8.3.2.1 Tensile Strength Estimation

Tensile strength testing can be overlooked for small engineering projects, possibly because the Hoek-Brown failure criteria determines the tensile strength for the GSI method automatically, based on Equation (8.1) (Hoek et al. 2002), and does not consider it an input property. The material constants, \( s \) and \( m_b \), are determined by the methods defined by Hoek et al. (2002) from triaxial tests:

\[
T = -\frac{sUCS}{m_b}
\]  

(8.1)

When reliable tensile testing data are unavailable, an estimate can be made using Equation (8.2), from Griffith’s theory (1924), as Diederichs (1999) indicated that this theory is consistent with the damage initiation threshold and the damage initiation and peak strength are coincident in tension.

\[
T_{\text{grif}} = \frac{CI}{\beta} ,
\]  

(8.2)

where \( \beta = 8 \) according to the original Griffith theory (1924) and can be as high as 12 according to the modified formulation of Murrell (1963) and Jaeger and Cook (1971). Perras and Diederichs (2014) showed that \( \beta = 12 \) is an absolute lower bound and \( \beta = 8 \) is a conservative average when comparing the available literature data for CI and T.

Perras and Diederichs (2014) showed that the BTS method overestimates the true tensile strength, as compared with a Direct Tensile Strength (DTS) test. To reduce the BTS value to the
equivalent DTS value a factor $f$ was determined ($DTS = f \cdot BTS$), which was found to be 0.9 for metamorphic, 0.8 for igneous, and 0.7 for sedimentary rocks.

The mean, standard deviation, and minimum and maximum values of CI, CD, UCS, and the various tensile strength values for the granite, limestone, and shale data sets are presented in Table 8.1. These parameters are needed to determine which modelling method is appropriate, GSI or DISL, and to calculate the input parameters for the numerical models.

Table 8.1: Mean, standard deviation, minimum and maximum values for the granite, limestone, and shale properties used in the numerical modelling. CIU\_AE = upper bound CI from AE measurements, CIL\_AE = lower bound, CI\_TD = CI from transducer measurements. $T=CI/8$, $T=CI/12$, $T=0.7BTS$ based on Perras and Diederichs (2014).

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Property</th>
<th>Mean</th>
<th>Standard Deviation</th>
<th>Min</th>
<th>Max</th>
</tr>
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<td>UCS (MPa)</td>
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<td>187</td>
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<td></td>
<td>CI (MPa)</td>
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<td>96</td>
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<tr>
<td></td>
<td>CD (MPa)</td>
<td>206</td>
<td>17</td>
<td>167</td>
<td>238</td>
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<td>-12.0</td>
<td>-21.5</td>
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<tr>
<td></td>
<td>CIU_AE (MPa)</td>
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<tr>
<td></td>
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<tr>
<td></td>
<td>CD (MPa)</td>
<td>32</td>
<td>12</td>
<td>15</td>
<td>50</td>
</tr>
<tr>
<td></td>
<td>$T=CI/8$ (MPa)</td>
<td>-2.4</td>
<td>0.8</td>
<td>-0.9</td>
<td>-3.7</td>
</tr>
</tbody>
</table>
8.4 Numerical Model Setup

Ductile failure or rock mass strain weakening can be simulated using the Hoek-Brown criteria (Hoek et al. 2002) and the GSI approach (Marinos et al. 2005) for rock masses and is widely used since their introduction in 1980 (Hoek and Brown 1980) and 1994 (Hoek 1994), respectively. This approach works well for rock masses which have some degree of jointing, which causes the rock mass strength to be lower than the intact laboratory properties. The joints allow the rock mass to behave in a shear-dominated manner.

For brittle rock masses, the degree of jointing is minimal (GSI > 75) and the behaviour is extensile-fracture dominated. The extensile fractures develop under compressive loading near the boundary of an excavation. As previously mentioned, Diederichs (2007) developed a simplified constitutive model approach, DISL, to represent the boundary sensitivity of brittle fracture. Depending on the constitutive model employed, the process of stress shedding and localization evolves differently. Determining the correct constitutive criteria is an important step in the design process.

8.4.1 Modelling Method Selection

Diederichs (2007) suggested a method to determine when GSI or DISL modelling methods should be conducted. This determination is made by using the rock mass GSI value and the strength ratio (UCS/T). For the purposes of this study, a GSI value of >80 has been used for all the rock masses being modelled. Based on this assumption and the mean properties presented in Table 8.1, the DISL approach is the most appropriate method to use for the numerical simulations. This approach is particularly well suited for finite element software which only allows peak and residual failure envelopes to be entered, such as Phase2 by Rocscience, which was
used for this research. The DISL approach and the influence on the EDZ dimensions are the main focus of this paper, along with dimension sensitivity.

8.4.2 Numerical Input Parameters

Based on the available data, the generalized Hoek-Brown failure criteria input parameters, \( m_p, s_p, \) and \( s_r \), for use with the DISL method are calculated using the equations shown in Table 8.2. The models were constructed to represent circular shaft excavations, with a radius of 3.25 m.

Two different in-plane (KHh) stress ratios of 1.5 and 2.0 were used. As Martin et al. (1999) suggested that brittle failure initiates at a \( \sigma_{\text{max}} / \text{UCS} \) ratio greater than 0.4 ± 0.1, where \( \sigma_{\text{max}} = 3\sigma_1 - \sigma_3 \), the depths of the models were adjusted to get a range of \( \sigma_{\text{max}} / \text{UCS} \) of 0.4 to 2. This range corresponds to a \( \sigma_{\text{max}} / \text{CI} \) ratio of 1 to 3. For the rock properties used in this paper (Table 8.1), the resulting depth ranges using a \( K_0 \) ratio of 2.5 were as follows:

- 1000–2345 m for the granite,
- 420–785 m for the limestone, and
- 200–375 m for the shale.

**Table 8.2: The equations to determine the DISL model input parameters, after Diederichs (2007).** The parameters \( a, s, \) and \( m \) are material constants (Hoek et al. 2002) and the subscripts \( p \) and \( r \) stand for peak and residual, respectively.

<table>
<thead>
<tr>
<th>Input Parameter</th>
<th>Value / Equation</th>
<th>Input Parameter</th>
<th>Value / Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>ap</td>
<td>0.25</td>
<td>ar</td>
<td>0.75</td>
</tr>
<tr>
<td>sp</td>
<td>( \left( \frac{CI}{UCS} \right)^{\frac{1}{a_p}} )</td>
<td>sr</td>
<td>0.001</td>
</tr>
<tr>
<td>mp</td>
<td>( s_p \left( \frac{UCS}{</td>
<td>T</td>
<td>} \right) )</td>
</tr>
</tbody>
</table>
The input parameters of the three models were determined by dropping 5% of the data (top 2.5% and bottom 2.5%) and determining the minimum, mean, and maximum values of the adjusted data. This eliminates the tails of the distribution and therefore the outliers which do not represent the general intact rock properties.

For the DISL input, the UCS and CI are taken as a pair from each test, such that as the UCS increases, the CI generally increases as well. As the input data come from real laboratory test results when the data are sorted from lowest to highest by UCS, some of the CI values are not in a lowest to highest order. The reason that the UCS and CI are taken as pairs is to show the impact of the natural variability of the input data on the resulting model output. It was decided to treat each test as a model input instead of only considering the mean input data. The minimum and maximum values, from the adjusted data, are assumed to represent the worst and best cases. The corresponding DISL composite envelop limits for the three rock types are plotted in Figure 8.3. The mean failure envelop splits the difference between the outer limits of each rock type. The mean input properties would result in the prediction of a single dimension for each damage zone. The purpose of this paper is to examine the range of variability in the damage dimensions based on the range of variability in the input data.

8.5 Numerical Results
To examine the variability in the numerical results, it is first necessary to establish a standard method of determining the dimensions of the EDZs. To understand which output values can be consistently used as numerical indicators of the EDZs, the damage zones were mapped to the DISL failure envelop conceptually, as shown in Figure 8.4.
8.5.1 Numerical Indicators of the EDZs

Conceptually the different EDZs were mapped to the constitutive (DISL) failure envelopes to understand the expected model behaviour. When the stress path of an element around the simulated tunnel changes in response to the excavation, this element is considered as part of the EIZ, as long as the stress path remains below the peak envelope. When the stress path crosses the peak envelope, the element yields plastically and the residual curve becomes the governing stress envelop, dictating the maximum allowable stress that an element can carry. If the stress path crosses the peak envelope in tension or in the spalling region, then this zone is considered to be part of the HDZ, and the element can be considered to undergo strain-weakening.

Figure 8.3: Composite DISL envelop limits for the three rock types used in the analysis.
Figure 8.4: Damage zones mapped to the peak and residual envelopes of the conceptual DISL approach of Diederichs (2007).

If the stress path crosses the peak envelope in the area where the residual envelope allows more stress to be carried by the element, it is considered to undergo strain-hardening. In this region, the elements damage zone indicator can range from EDZo close to the peak envelope to EDZi near the residual envelope.

An example of the mean limestone case is used to illustrate the difference between the outer and inner EDZs. The different points in Figure 8.5 represent different stages of the model used to simulate three-dimensional excavation advance in two dimensions, using a gradually decreasing distributed load around the excavation surface. The distributed load is gradually
decreased from original stress state down to 0.5% of the original stress state over 11 stages. In the final stage, the 12th, the distributed load is removed.

Three stress paths are shown (Figure 8.5) with respect to the DISL failure envelopes, one taken to represent the HDZ, the EDZi, and the EDZo, at distances from the excavation surface of 0.0 m, 0.6 m, and 1.4 m, respectively. Also shown is the background stress field. The stress path analysis was used to validate and update the conceptual model by demonstrating that the volumetric strain reversal, from contraction to extension, is an important indicator of the behaviour of the rock around the tunnel model. This transition occurs when the stress path crosses in the intersection of the peak and residual envelops (Figure 8.5) and this point has been called, $\sigma_{3crt}$. All stress paths cross the failure envelope to the right of $\sigma_{3crt}$. For the stress path at the excavation surface, stresses exceed the peak DISL curve, causing the stresses to follow the residual curve (higher strength when to the right of $\sigma_{3crt}$). When the stresses at the surface pass $\sigma_{3crt}$, there is a deviation in the linear stress path at 0.6 m from the excavation surface.

The comparison of Figure 8.5a and 8.5b show that this point in the stress evolution corresponds to the extensile strain at the excavation surface, resulting in stress shedding to the surrounding elements. This shedding also occurs when the stresses at 0.6 m away from the excavation surface fall below $\sigma_{3crt}$. At 1.4 m from the excavation surface, the stress shedding to 0.6 m is also evident by a deviation from the linear stress path. The EDZi is associated with the extensile strain, which occurs when the minimum principal stress falls below $\sigma_{3crt}$. However, for the limestone case, at 1.4 m (near the start of the EDZo), the stresses remain above $\sigma_{3crt}$ and the strains remain in contraction. Realistically, this confinement would inhibit fracture propagation; therefore, the damage in the EDZo is considered to be distributed and unconnected.
Figure 8.5: Stress path (a) and volumetric strain (b) evolution as the internal pressure on the excavation wall is reduced to simulate three-dimensional excavation advance in two-dimensions using Phase2 by Rocscience. The in-plane stress ratio, $KHh = 2.0$. 

$\sigma_3 (\text{MPa})$ vs $\sigma_1 (\text{MPa})$

$\sigma_3 (\text{MPa})$ vs Volumetric Strain

$\sigma_3 (\text{MPa})$ vs $\sigma_1 (\text{MPa})$
Conceptually, as indicated in Figure 8.4, as the stress path approaches the residual curve, there is a transition to an indication of EDZi. A gradational transition from EDZo to EDZi is consistent with the concept of crack initiation, which starts as random damage building to systematic, but unconnected, damage, and eventually building until the cracks begin to interact.

Based on the work of Perras et al. (2010, 2012) and the model results from this paper, the yielded elements, volumetric strain, and principal stress concentrations were found to be the best indicators for determining the dimensions of different EDZs. These values are plotted against the distance from the excavation surface for the mean granite, limestone, and shale models in Figure 8.6 with the appropriate EDZ dimensions indicated. The values were measured along a line which passes from the centre of the excavation through the deepest yield zone away from the excavation surface (parallel to $\sigma_3$). The contours of the minimum principal stress and volumetric strain with highlights of the EDZ limits are shown for each mean model case as an inset image in Figure 8.6. The modelling presented here is for circular excavations only and the results should be used with caution for other excavation shapes and for stress regimes outside of those evaluated.

Plastic yielding indicates that the peak elastic properties have been exceeded, which results in the onset of distributed damage in the rock mass. Inspection of Figure 8.6 shows that the volumetric strain is in contraction and still increasing, which indicates that despite damage occurring, it is in a confined state inhibiting grain-scale fractures from propagating. The outer limit of plastic yielding therefore corresponds to the outer limit of the EDZo.

The start of extensile strain, as discussed previously, is used as the indicator for the start of the EDZi. In some cases, such as when the stress ratio ($KH_h$) is close to 1, no contraction strain occurs before extension strain, because of the uniform shape of the plastic yield zone (Perras et al. 2010). Therefore, an indicator based on peak contraction strain would not apply in all cases.
Figure 8.6: Numerical results for the mean a) granite, b) limestone, and c) shale models showing the change in properties over the distance from the excavation surface and how they relate to the damage zones. Inset model results, from Phase2, of the minimum principal stress ($\sigma_3$) and volumetric strain ($\varepsilon_v$) contours with main model information shown.
The volumetric strain reversal point is also consistent with a decrease in the confining stresses and the steepest slope of the distance versus maximum shear strain in Figure 8.6, which indicates for a rock mass that the damage can propagate from grain to grain at the micro scale. As the extensile volumetric strain continues to increase, it reaches a maximum value, which coincides with minimum principal stresses and continued increase in shear strains. The HDZ limit is selected as the first point where \( \sigma_3 \) begins to increase from the level at the excavation boundary (zero). This indicates that the rock mass is beginning to be able to carry some load and therefore macro fractures are limited in length due to increasing confinement moving away from the excavation boundary.

The model contours, in Figure 8.6 insets, show that the nature of the damage zones are not divided by smooth transitions, but rather that the shape fluctuates. The maximum extent in each model was used to determine the dimensions of the EDZs. In the cases shown, it corresponds to the orientation of the principal stress orientations. However, if more complex geometries or conditions are modelled, this correspondence may not be the case. The following specific criteria (guidelines) were used to determine the dimensions of the EDZs reported in Table 8.3, based on one model for each laboratory test.

- The HDZ–EDZi transition was taken as the first point where \( \sigma_3 \) increases from the value at the excavation surface and either maximum or rapidly decreasing extensile or shear strain moving away from the excavation surface.
- The EDZi–EDZo transition was taken as the start of extensile volumetric strain.
- The EDZo–EIZ transition was taken as the start of plastic yielding.

The stress evolution analyses helped to confirm the observations from the numerical models, to better understand the volumetric strain criteria and were used to develop the guidelines presented for determining the dimensions of the different EDZs.
Table 8.3: The minimum, mean, and maximum dimensions of the EDZs for the various cases modelled.

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>In Plane Stress Ratio</th>
<th>Zone</th>
<th>DISL Cases (m)</th>
<th>GSI Cases (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Min</td>
<td>Mean</td>
</tr>
<tr>
<td>Granite</td>
<td>KHh = 1.5</td>
<td>HDZ</td>
<td>0.0</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>EDZi</td>
<td>0.3</td>
<td>0.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>EDZo</td>
<td>0.4</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>KHh = 2.0</td>
<td>HDZ</td>
<td>0.1</td>
<td>0.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>EDZi</td>
<td>0.1</td>
<td>0.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>EDZo</td>
<td>0.3</td>
<td>1.0</td>
</tr>
<tr>
<td>Shale</td>
<td>KHh = 1.5</td>
<td>HDZ</td>
<td>0.6</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>EDZi</td>
<td>0.7</td>
<td>1.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>EDZo</td>
<td>1.2</td>
<td>2.3</td>
</tr>
<tr>
<td></td>
<td>KHh = 2.0</td>
<td>HDZ</td>
<td>0.2</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>EDZi</td>
<td>0.3</td>
<td>0.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>EDZo</td>
<td>0.3</td>
<td>1.5</td>
</tr>
<tr>
<td>Limestone</td>
<td>KHh = 1.5</td>
<td>HDZ</td>
<td>0.2</td>
<td>0.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td>EDZi</td>
<td>0.3</td>
<td>1.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>EDZo</td>
<td>0.3</td>
<td>1.8</td>
</tr>
<tr>
<td></td>
<td>KHh = 2.0</td>
<td>HDZ</td>
<td>0.0</td>
<td>0.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>EDZi</td>
<td>0.3</td>
<td>1.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>EDZo</td>
<td>0.4</td>
<td>1.7</td>
</tr>
</tbody>
</table>

8.5.2 Numerical Dimensions of the EDZs

The empirical plot from Martin et al. (1999) can be used as a preliminary assessment of the maximum depth of brittle failure around underground excavations. Diederichs (2007) adapted this empirical plot to consider CI, as it has been shown to be an important rock property to describe the brittle rock mass behaviour underground. This plot is based on case studies in which the maximum damage depth was investigated through intense scaling. The data set applies to tunnel wall stress levels up to 2.5 times CI, with few cases at the upper end. Using modelling and the
interpretation methods in this paper, it is possible to delineate HDZ, EDZi, and EDZo trends in a similar manner to the empirical approach of Martin et al. (1999) and Diederichs (2007).

The numerical dimensions of the EDZs for the granite, limestone, and shale are shown in Figure 8.7, along with the empirical limits for the in-plane stress ratio of $KH_h = 2.0$ model results. From each model computed three points were produced for Figure 8.7, one for each damage zone (EDZo, EDZi, and HDZ). The numerical results indicate that the maximum empirical limit corresponds to the EDZo to a $\sigma_{max}/CI$ ratio of approximately 2. Above this value, the linear empirical limits overpredict the dimensions compared to the non-linear numerical results for all three rock types. Similarly, the EDZi would be over predicted if the minimum empirical limit were used when compared with the numerical results. The HDZ should not be predicted from the empirical approach, because the empirical approach is based on the maximum overbreak and intense scaling. The HDZ has a similar curvature, with a shallower ‘slope’ to the EDZo and EDZi, although there is more scatter in the HDZ numerical results.

The best fit equations for each of the damage zones for each rock type are shown in Figure 8.7, with the corresponding $R^2$ values. There are some differences between the multiplier and exponent for each rock type. These differences will be discussed in more detail later in the paper.

The plots in Figure 8.7 could be used to predict the dimension of the damage zones and adjust the radius of the excavation to minimize the damage dimensions. The variability of the dimensions has also been captured based on the variability of the input properties in this numerical study.
Figure 8.7: All model results for the a) granite, b) limestone, and c) shale with an in plane stress ratio, $KHh = 2.0$. The best fit equations for the EDZs and the $R^2$ values are indicated.
8.5.2.1 Regression Analysis

A non-linear regression analysis was conducted to determine the best fit equation to the damage zone dimension for each rock type and stress scenario tested. The results are shown in Table 8.4 for all cases and examples for each rock type with a KHh = 2.0 are shown in Figure 8.7. The form of the equation was first established by Perras et al. (2012), where the multiplier and the exponent in the general form of the equation (Equation 8.3) are B and D, respectively:

\[
R / a = 1 + B \left(\frac{\sigma_{\text{max}}}{CI} - 1\right)^D
\]  

(8.3)

The non-linear regression was used to determine the best fit B and D values for each case. This determination was made following a similar method as outlined by Langford and Diederichs (2013). Generally, the EDZo has the least scatter with the highest R\(^2\) values, as shown in Figure 8.7. The HDZ has the lowest R\(^2\) values. Generally speaking, the multiplier B is largest for EDZo and smallest for the HDZ. The same can be said for the exponent D, with the one exception being the granite case, in which the HDZ has the highest D parameter. This exception exists because of the large number of numerical results in which the HDZ is very small relative to the excavation dimension, and so the curve fit approaches a linear fit (D = 1).

As engineering projects become increasingly more challenging, understanding the range of the expected behaviour and trying to determine the most likely dimensions of the failure zone become more important, which in this case is the EDZ dimensions.

8.6 Statistical Evaluation

The model results were analyzed using three methods to produce cumulative probability distributions. In a first-pass analysis, the model results corresponding to the minimum, mean, and maximum UCS tests were evaluated statistically using the 3σ rule, which implies that the best or worst case lies 3 standard deviations either side of the mean. This assumption is a reasonable first
assumption, as the input data are normally distributed. The 3σ rule would imply that the results are also normally distributed. This implication allows for a standard deviation to be determined from the range (of the dimensions for this study), and this standard deviation can be used to create a cumulative distribution. This approach was compared with selecting the models which correspond to the minimum, mean, and maximum CI tests.

Table 8.4: Multiplier (B) and exponent (D) for the best fit EDZ dimension curves.

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>KHi</th>
<th>Zone</th>
<th>B</th>
<th>D</th>
<th>$R_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Granite</td>
<td>1.5</td>
<td>EDZo</td>
<td>0.62</td>
<td>0.58</td>
<td>0.81</td>
</tr>
<tr>
<td></td>
<td></td>
<td>EDZi</td>
<td>0.41</td>
<td>0.53</td>
<td>0.78</td>
</tr>
<tr>
<td></td>
<td></td>
<td>HDZ</td>
<td>0.09</td>
<td>0.62</td>
<td>0.48</td>
</tr>
<tr>
<td>Granite</td>
<td>2.0</td>
<td>EDZo</td>
<td>0.58</td>
<td>0.65</td>
<td>0.81</td>
</tr>
<tr>
<td></td>
<td></td>
<td>EDZi</td>
<td>0.36</td>
<td>0.62</td>
<td>0.64</td>
</tr>
<tr>
<td></td>
<td></td>
<td>HDZ</td>
<td>0.11</td>
<td>0.85</td>
<td>0.45</td>
</tr>
<tr>
<td>Limestone</td>
<td>1.5</td>
<td>EDZo</td>
<td>0.66</td>
<td>0.63</td>
<td>0.95</td>
</tr>
<tr>
<td></td>
<td></td>
<td>EDZi</td>
<td>0.43</td>
<td>0.58</td>
<td>0.93</td>
</tr>
<tr>
<td></td>
<td></td>
<td>HDZ</td>
<td>0.18</td>
<td>0.34</td>
<td>0.42</td>
</tr>
<tr>
<td>Limestone</td>
<td>2.0</td>
<td>EDZo</td>
<td>0.58</td>
<td>0.58</td>
<td>0.89</td>
</tr>
<tr>
<td></td>
<td></td>
<td>EDZi</td>
<td>0.36</td>
<td>0.49</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td></td>
<td>HDZ</td>
<td>0.12</td>
<td>0.33</td>
<td>0.12</td>
</tr>
<tr>
<td>Shale</td>
<td>1.5</td>
<td>EDZo</td>
<td>0.71</td>
<td>0.59</td>
<td>0.93</td>
</tr>
<tr>
<td></td>
<td></td>
<td>EDZi</td>
<td>0.49</td>
<td>0.55</td>
<td>0.91</td>
</tr>
<tr>
<td></td>
<td></td>
<td>HDZ</td>
<td>0.20</td>
<td>0.52</td>
<td>0.67</td>
</tr>
<tr>
<td>Shale</td>
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<td>EDZo</td>
<td>0.66</td>
<td>0.59</td>
<td>0.97</td>
</tr>
<tr>
<td></td>
<td></td>
<td>EDZi</td>
<td>0.39</td>
<td>0.59</td>
<td>0.93</td>
</tr>
<tr>
<td></td>
<td></td>
<td>HDZ</td>
<td>0.15</td>
<td>0.68</td>
<td>0.75</td>
</tr>
<tr>
<td>All</td>
<td>All</td>
<td>EDZo</td>
<td>0.61</td>
<td>0.59</td>
<td>0.87</td>
</tr>
<tr>
<td></td>
<td></td>
<td>EDZi</td>
<td>0.37</td>
<td>0.50</td>
<td>0.69</td>
</tr>
<tr>
<td></td>
<td></td>
<td>HDZ</td>
<td>0.15</td>
<td>0.65</td>
<td>0.45</td>
</tr>
</tbody>
</table>
The results are shown in Figure 8.8 for the limestone models with a KHh = 1.5. In comparing the model results selected by UCS and CI, the distribution based on the UCS tests yields a more conservative cumulative probability distribution. Although, at a probability of 90% the dimensions are within 0.1 m of each other, which is a reasonable resolution for predicting the dimensions of the EDZs. However, when comparing the distributions based on the input values with the output results, there is a larger difference, particularly in the tail regions.

For the limestone case, with a KHh = 1.5, the probability distribution based on the statistical output (minimum, mean, and maximum) dimensions were determined (Figure 8.8). Using all the model results allows for a better estimate to be made of the true mean and standard deviation of the dimensions of the EDZs, but it still weights each model equally (i.e., normally distributed). There is an increase in the predicted EDZi using all the results by approximately 0.2–0.3 m over using the UCS or CI input value models to determine the distribution.

It is possible that the dimensions of the EDZs are not normally distributed over the entire range of parameters evaluated in this study. If such is the case, then the response surface method can be used, as it weights each model result based on the distance from the mean (geometrically) rather than weighting each result in a linear fashion, as is the case when the results are assumed to be normally distributed.

When the response surface method distribution is compared with that from the normally distributed model output results, they are seen to be very similar (Figure 8.8). This similarity suggests that, in fact, the results are normally distributed over the range of the input values used in the numerical modelling.

Using this finding, each rock type was evaluated based on all the models computed for each in-plane stress ratio. The results are shown in Figure 8.9 for the HDZ, EDZi, and EDZo for
each rock type. As expected, the probable damage zone dimensions are smallest for the granite, with steep probability distributions for the different damage zones. This relation indicates less variability in the dimensions of the damage zones for the granite over the other rock types. For the granite, the HDZ distribution is the same for both in-plane stress cases. The probable dimensions for the EDZi and EDZo are larger for the higher (KHh = 2.0) in-plane stress case. For both stress cases, the distribution trends for all the damage zone dimensions are similar.

Figure 8.8: Cumulative probability distributions for the EDZi produced from the models of the limestone data set with an in plane stress ratio of KHh = 1.5. Input model results are based on the minimum, mean, and maximum UCS or CI values. Output model results consider the minimum, mean, and maximum EDZi from all the models.
The limestone case presents more variability in the dimensions, and the lower in-plane stress ratio results indicate larger dimensions. This finding is the opposite of the granite results because the granite model damage zone dimensions are similar for both in-plane stress ratios. With the limestone models, a larger difference exists between the damage zone dimensions. Despite this difference with the granite results, the slopes of the limestone distributions remain similar when comparing the same damage zone distribution between the two stress scenarios.

The shale case presents a large range of variability which causes the slope of the distribution to change as the stress scenario changes. This relation is most notable for the EDZo, which shows a large difference in the predicted damage dimensions between the two stress regimes. Similar to the limestone case, the KHh = 1.5 model results are larger than the KHh = 2.0.

Using the cumulative probability distributions, the dimensions can be stated with a degree of confidence. For example, at the 90% confidence interval for the granite case (Figure 8.9a), the dimensions are 0.3, 0.7, and 1.0 m, for the HDZ, the EDZi, and the EDZo, respectively.

Similarly, in Figure 8.9b, there is 90% confidence that the HDZ, EDZi, and EDZo have dimensions of 0.7, 1.5, and 2.3 m, respectively, for the KHh = 1.5 limestone case, and in Figure 8.9c, the dimensions are 0.7, 1.8, and 5.4 m, for the shale.

Langford and Diederichs (2011), who evaluated an excavation design with multiple statistical methods, concluded that although these additional runs increase the work load, a more reliable result is produced without having to conduct a full Monte Carlo simulation. Additional runs, to determine how the cumulative distribution curve will change in the tail regions, could be evaluated using synthetic input parameters that are within the given range of the testing results. These runs would help to establish more reliable estimates of the mean dimensions.
Figure 8.9: Cumulative probability distributions using the 3σ rule for the a) granite, b) limestone, and c) shale models for both in plane stress ratios, based on the output minimum, mean, and maximum results.
However; the methodology presented here remains the same and can be useful for optimizing cut-off depths or determining if different excavation dimensions should be evaluated, for example. In many engineer design contracts, there is little time in the budget for additional numerical analysis. It is therefore important to understand what input parameters have the most impact on the model outcome so as to focus sensitivity analysis for design purposes.

8.7 Dimension Sensitivity
Plotting the resulting normalized dimensions of the EDZs, as in Figure 8.7, against the maximum tangential stress ($\sigma_{\text{max}}$) normalized against CI accounts directly for the variation in the stress and indirectly for the compressive strength used in the numerical models. As previously discussed, the stress scenario was varied by adjusting the depth used to determine the vertical stress ($\sigma_z$) and by adjusting the in-plane stress ratio ($KHh$). Similarly, the CI is a key input property for the DISL approach, which is a compressive strength threshold in this case. There is also a relation between UCS and CI, so variations in UCS should result in similar variations in CI, and these variations are captured through the normalization by CI.

8.7.1 Upper and Lower CI
As discussed previously, the different methods of determining CI can result in slightly different values for the same test sample. These values are determined as upper and lower bound values. Both upper and lower AE CI values are generally lower than the transducer CI values.

Numerically, a series of runs were evaluated wherein the only change was the CI input value and the stress scenario. All three possible CI values, as reported (Gorksi et al. 2009-2011 and Ghazvinian et al. 2012b) for the limestone, were used. The results, in Figure 8.10, show that generally there is only a small difference in the best fit curves (2%–4%) for $\sigma_{\text{max}}$/CI values less than 2.5, with the exception of the HDZ. The best agreement between the different fits is for the...
EDZ₀, and the agreement becomes less for the damage zones closer to the excavation surface. Considering the stress path evaluation (Figures 8.4 and 8.5) for the elements closest to the excavation surface, the path crosses near \( \sigma_{3\text{cr}} \). If all other input properties (UCS and T) remain the same and only CI changes, which is the intercept of the peak curve with the y axis (see Table 8.2 for governing equations), than the slope of the peak curve remains the same, but the magnitude of the stress which an element can carry before exceeding peak changes. With a lower CI value the elements can carry less stress before yielding and therefore result in a larger HDZ. However, for the inner and outer EDZ, a lower CI allows elements closer to the excavation surface to remain above \( \sigma_{3\text{cr}} \) because it shifts to the left with lower CI values if all other inputs remain the same.

As the results indicate for real variations between upper and lower CI values, as measured in the laboratory, this finding only has a significant impact at higher confining stresses and on the HDZ. In most engineering situations where flow through the damage zones is to be minimized, the most effective way to deal with HDZ zone is to remove it, and therefore the impact on the project may be slightly more or less material to excavate.

There should be minimal impact on the EDZᵢ if only the loose material is removed, and excessive scaling is minimized such that the stress in the rock mass is not changed, which could change the depth of the EDZᵢ. There is more influence from the tensile strength used in the numerical approach on the inner and outer EDZ than the CI value used.

### 8.7.2 Influence of Tensile Strength

The tensile strength is also an important input property for the DISL approach; however it is absent from the normalization method used to evaluate the damage dimensions. Several methods of estimating tensile strength, the measured BTS, and reduced BTS were used, based on the
findings of Perras and Diederichs (2014). The tensile strength variations were applied to the limestone models with a KHh ratio of 2.0, and the results are shown in Figure 8.11a for the EDZs.

The variation of the dimensions of the EDZs when only the tensile strength is adjusted (for the same $\sigma_{\text{max}} / \text{CI}$) has a large influence when the $\sigma_{\text{max}} / \text{CI}$ ratio is large, and the largest deviation is shown to be for the EDZo (Figure 8.11a). Generally, good agreement exists between the dimensions of the EDZs and between the models wherein the tensile strength is estimated based on CI using a factor of 8 or 12.

![Figure 8.10: Sensitivity analysis using different CI values for the limestone data set with an in plane stress ratio of KHh = 2.0. CI_TD is for transducer measurements (values from Gorksi et al 2009, 2010, 2011), CIU_AE is an upper bound CI based on AE measurements, and CIL_AE is a lower bound (values from Ghazvinian et al. 2012b). $R^2$ values are indicated for each set and damage zone.](image-url)
The largest tensile strength, the mean BTS, results in the largest predicted dimensions. This observation is counterintuitive, however; when the different peak failure envelopes are examined (Figure 8.11b), the largest tensile strength results in the shallowest slope of the peak failure envelop in compression. Thus, for the same stress path, for example at $\sigma_{\text{max}} / \text{CI} = 3$ in Figure 8.11b, the rock mass can carry more load before failure in compression at lower tensile strengths. This is not a reasonable relationship and points to the importance of using the correct tensile strength as an input. Reducing the mean BTS, following the suggested relation (Perras and Diederichs 2014), to a DTS value by a factor of 0.8 ($\text{DTS} = 0.8 \text{ BTS}$) yields dimensions which are closer to those determined using the estimated tensile strength methods. At a $\sigma_{\text{max}} / \text{CI}$ of 2.5 there is clustering of the results around a very similar EDZi dimensions and around a similar HDZ dimension. This is perhaps because the CI values are not increasing linearly with the UCS values, since real data has been used. Generally there is an increasing difference between each of the tensile strength methods with increasing confinement.

The authors recommend that DTS values be determined whenever possible and in close spatial proximity to UCS samples. The numerical results suggest that a good first approximation, in the absence of tensile test results, is to estimate the tensile strength using $\text{CI} / 8$, as it generally gives a conservative estimate of the dimensions of the EDZs. If BTS results are to be used, they should be reduced to equivalent direct values, as Perras and Diederichs (2014) suggested that BTS values are generally greater than DTS. The higher tensile strength values, in fact, yield larger EDZs when the failure mechanism is compression induced.

### 8.7.3 Mesh Dependency

When determining the dimensions of the EDZs, the size of the mesh elements is important to consider, as the resolution of the mesh size will be the limiting dimension resolution possible. In
the models used in this paper, the mesh elements measured 0.06 m at the excavation surface, and the dimensions of the EDZs were determined to the closest 0.1 m. Where the limit fell exactly in between an increment of 0.1 m, the value was taken to the closest 0.05 m. The DISL method is sensitive to the mesh geometry, as it is a localization problem.

Localization of shear strains occurs within the plastic yield zone and can create shear bands. Needleman (1988) indicated that such localization occurs for rate-dependent solids for both quasi-static and dynamic loading conditions. Varas et al. (2005) analyzed a series of models in which localization was observed and found that the displacements were influenced the most when using a fine mesh. They also concluded that the effects are irrelevant when a large mesh size was selected, between 0.1 and 0.05 of the radius.

In the present study, the dimensions of the EDZs were also found to be relatively insensitive over a similar range, as shown in Figure 8.12, for a 3-noded triangular mesh. All the damage zone dimensions, up to a mesh dimension to excavation radius ratio of 0.03, are similar with this mesh style. For a 6-noded triangular mesh, there is increasing damage dimensions as the mesh size is decreased, even for coarse meshes. The 6-noded triangular mesh reduces the stiffness of the rock mass over that of a 3-noded mesh, so the results are not directly comparable.

Because the focus of this paper is on the variability introduced by the natural variability of the input properties, a single mesh style was selected for all models. The mesh dimension to excavation radius ratio of 0.02 was selected to optimize the computational time and the maximum damage zone dimensions. Below 0.02, the computational time increased significantly, with only a small increase in the damage zone dimensions, as indicated in Figure 8.12.
8.8 Discussion and Conclusions

The natural variability of intact rock properties has been considered by explicitly incorporating each laboratory test into a numerical model and evaluating the variability in the dimensions of the EDZs. The results of all the numerical simulations (each rock type, stress scenarios, etc.) have been compiled, and the mean fit equations for each damage zone are shown in Figure 8.13.

Figure 8.11: The influence of different mesh dimensions and styles (3- and 6-noded triangles) on the damage zone dimensions, with an inset image showing the mesh arrangement and the mesh dimensions discussed in the text at the excavation surface.
Figure 8.12: Combined model results showing the mean damage zone equation with the $R^2$ values for a) the EDZo, b) the EDZi, and c) the HDZ, including prediction intervals at 68% and 95% of the results.
The best fit is for the EDZo, with an $R^2$ of 0.87, which also shows the least spread in the prediction intervals (Figure 8.13a). The EDZi also has a reasonably good fit ($R^2$ of 0.69); however, the prediction intervals are not evenly distributed about the mean curve (Figure 8.13b). Similarly, for the HDZ, the prediction intervals are not evenly distributed because they are constrained below the mean by no damage ($r/a = 1$), as shown in Figure 8.13c. The prediction intervals provide an estimate of the interval within which future data points will fall, given the existing data and the form of the mean fit (Langford 2013). This differs from confidence intervals which provide an estimate of the distribution of the existing data.

Considering the different groupings of modelling scenarios, the multiplier and exponent in Equation (8.3) can have a wide range in some cases (see Table 8.4). The range is generally smaller for the EDZo and larger for the HDZ. There is a more consistent value of the multiplier (B) for all model scenarios, with a difference between the maximum and minimum values ranging between 0.11 and 0.13. The exponent (D) has a wider difference, ranging from 0.07 to 0.52. The final forms of the mean equations to describe the damage zone dimensions are as follows:

\[
\text{EDZo} / a = 1 + 0.6(\pm0.07) (\sigma_{\text{max}}/\text{CI} - 1)^{0.6(\pm0.04)} \\
\text{EDZi} / a = 1 + 0.4(\pm0.07) (\sigma_{\text{max}}/\text{CI} - 1)^{0.5(\pm0.07)} \\
\text{HDZ} / a = 1 + 0.2(\pm0.06) (\sigma_{\text{max}}/\text{CI} - 1)^{0.7(\pm0.25)}. 
\]

The delineation of the dimensions of the EDZs is an important factor in the design of underground excavations, particularly those where the influence on the porosity and permeability of the surrounding rock mass is to be minimized, such as in the case for underground nuclear waste storage. High-quality geotechnical data from investigations improve the engineering understanding of the rock mass behaviour and with a large sample set, allows for statistical
methods to be employed to, at a minimum, determine the minimum, mean, and maximum numerical input properties. These properties can then be used in numerical continuum codes to delineate the dimensions of the EDZs. The numerical approach investigated in this paper was determined to be most sensitive to the tensile strength used as an input. The variation in the EDZs can be used to further refine the design of the underground excavation, considering the likelihood of the dimensions. This method allows for optimizations of the excavation geometry, support, and, most crucially, the depth of cut-off structures in the case of permeability sensitive structures with a certain degree of confidence.

8.9 Acknowledgements
The work presented in this paper has been funded by the Natural Sciences and Engineering Research Council of Canada and by the Nuclear Waste Management Organization of Canada. The authors thank those who reviewed this work for their valuable discussion and suggestions. Particular thanks are given to those on the Queen’s Geomechanics Research Team in the Department of Geological Science and Geological Engineering for the valuable discussion and support in this research. A special thank you is due to Dr. Connor Langford for his assistance with the statistical evaluation of the numerical results.

8.10 References


Chapter 9: Underground Excavation Behaviour of the Queenston Formation: tunnel back analysis and forward shaft prediction

9.1 Abstract
The Niagara Tunnel Project (NTP) is a 10.1-km-long water-diversion tunnel in Niagara Falls, Ontario, which was excavated by a 7.2 m radius tunnel boring machine. Approximately half the tunnel length was excavated through the Queenston Formation, which locally is a red shale to mudstone. A large notch formed in the tunnel crown and invert during construction. Typical overbreak depths ranged between 2–4 m with a maximum of 6 m observed. Three modelling approaches were used to back analyze the brittle failure process at the NTP: the Damage Initiation and Spalling Limit (DISL) approach, the laminated anisotropy modelling (LAM) approach, and a ubiquitous joint (UBJT) approach. Analyses were conducted for three tunnel chainages: 3000, 3250, and 3500 m because the overbreak depth changes from 2 to 4 m in this 500 m length of the tunnel. All approaches could produce a notch of similar geometries to that measured at each chainage analyzed. The LAM approach was the most consistent at accurately capturing chord closure measurements. For the laminations, good agreement with the chord closure was achieved with joint normal to shear stiffness between 1–2. This understanding was applied to a shaft excavation in the Queenston Formation at the proposed Deep Geological Repository (DGR) site for low and intermediate level nuclear waste storage near Kincardine, Ontario. The maximum damage depth was 1.9 m; with an average of 1.0 m. Important differences are discussed between the tunnel orientation for NTP (parallel to bedding) and the DGR shaft.

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(perpendicular to bedding). The models show that the observed normalized depth of failure at the NTP would over-predict the depth of damage expected in the Queenston Formation at the DGR.

9.2 Introduction

The Queenston Formation is an extensive sedimentary layer in both the Appalachian and Michigan sedimentary basins of North America. It is exposed at the surface along the base of the Niagara Escarpment, as shown in Figure 9.1. It is an important raw material for the brick industry, and many civil engineering projects have been constructed on or in the Queenston rock mass. The most recently completed, the Niagara Tunnel Project (NTP), is a 7.2 m radius water diversion tunnel in the city of Niagara Falls, Ontario, Canada. Of the total 10.2 km length of the tunnel, approximately 5 km were excavated within the Queenston Formation. The tunnel gradient was shallow relative to the bedding dip throughout most of the tunnel.

In contrast, a shaft excavation perpendicular to the bedding is being proposed for access to and ventilation for a Deep Geological Repository (DGR) for Low and Intermediate Level Nuclear Waste storage in the Cobourg Formation. Extensive investigations have been conducted at the Bruce Nuclear Power Station for this DGR, where the Queenston Formation is approximately 73 m thick. These two projects are used to study the effect of the excavation orientation on the rock mass behaviour and to determine the influence of anisotropy on the damage zone dimensions.

9.2.1 The Niagara Tunnel Project

The NTP is a water diversion tunnel for hydropower generation. The tunnel diverts water from above Niagara Falls to the Sir Adam Beck (SAB) generating station, as shown in Figure 9.2a. The project decreased the amount of time that the available water for diversion exceeds the SAB...
capacity from 65% to 15% (Delmar et al. 2006). Excavation of the NTP began in August 2006 and was completed in May 2011. The project went into operation in March 2013.

The tunnel was excavated using a tunnel boring machine (TBM), and challenging overbreak conditions led to project delays. The overbreak was focused in the crown and invert as a result of the high horizontal stress ratio. The bedding reportedly dips 6 m/km (Yuen et al. 1992) and can be considered nearly horizontal at the scale of the tunnel. The overbreak in the section of the tunnel within the Queenston Formation typically reached 4 m in the high stress areas, with local maximums reaching 6 m, and presented significant tunnelling difficulties. Observations of the excavation performance were documented in Chapter 5 for the first 3.5 km.

Figure 9.1: Regional surface exposure of the Queenston and Georgian Bay Formations in Southern Ontario showing the NTP and DGR site locations (modified from Armstrong and Carter (2010).
Figure 9.2: An overview of the Niagara Tunnel Project showing a) the plane view of the tunnel in relation to existing infrastructure, b) a longitudinal cross section along the tunnel alignment showing the main geological units (modified from Perras et al. 2013), and c) the depositional setting of the Queenston across Ontario (after Brogley et al 1998).
The nearly horizontal tunnel alignment in the Queenston Formation (see Figure 9.2b), which closely parallels the bedding, contributed to the overbreak. Local variations in the strength, stress, and stiffness also contributed to changes in the overbreak geometry. As the deposition of the clastic material moved in a north-westerly direction, the rocks became finer grained, and the calcite content increased (Figure 9.2c). Fluctuating sea levels would cause local variations also.

The behaviour of the Queenston Formation from the NTP presents an opportunity to back analyze the deformations to determine the appropriate strength, stress, and anisotropic properties that give rise to numerical results similar to those measured at the tunnel (see Perras et al. 2013). This understanding is then applied to the DGR shaft excavation in the Queenston Formation for forward prediction.

9.2.2 The Deep Geological Repository

The Nuclear Waste Management Organization (NWMO) is proposing to construct a DGR 250 km northwest of the NTP. The proposed site is located below the site of the Bruce Nuclear Generating Station. The footprint is shown in Figure 9.3a in relation to the reactor buildings (Bruce A and Bruce B), and emplacement horizon is proposed to be in the Cobourg Formation (Figure 9.3b), at approximately 680 m below the ground surface. The project will include an access shaft with a radius of approximately 4 m and a slightly smaller ventilation shaft. A 200 m thick shale sequence, including the Queenston Formation, overlies the Cobourg and forms a regional aquitard. The shale formations provide a natural barrier between saline basin fluids and the overlying ground water resources near the surface.

As part of the regulatory approval process for the DGR, the Environmental Impact Statement, Preliminary Safety Report, and other supporting documents were submitted to the Canadian Nuclear Safety Commission Review Panel on April 14, 2011. For a detailed review of
the project and the geological setting, the reader is referred to the Descriptive Geosphere Site Model (Intera 2011) and the Geosythesis (NWMO 2011) reports.

This paper investigates issues related to the back analysis of the NTP, with the goal of understanding the key numerical inputs that reliably reflect the observed overbreak. Taking these findings, and accounting for site specific variations, numerical modelling is presented to assess the behaviour of the Queenston during shaft excavation.

Figure 9.3: Overview of the Bruce Nuclear site on the eastern coast of Lake Huron showing the location of a) the DGR footprint in relation to the Bruce Nuclear site and the reactors (Bruce A and Bruce B) and b) the geological stratigraphy (modified from NWMO 2011) with the emplacement horizon in the Cobourg at an elevation of 680 m below ground.
9.3 Geological Setting

The NTP is located in the Appalachian sedimentary basin, and the DGR is located in the Michigan sedimentary basin (Figure 9.1). The Appalachian basin is a back arc basin, and the Michigan basin is an inner cratonic basin. This means that there are more coarse grained sedimentary rocks and higher stresses in the Appalachian basin because of closer proximity to the fold and thrust belts of eastern Canada than the Michigan basin. The Michigan basin has more carbonates and evaporate deposits because of periods of isolation from the ocean which typical of inner cratonic basins. The isolation was caused by the Algonquin Arch, which is a high ridge in the Precambrian basement rock. Some sedimentary formations are truncated forming unconformities across the arch, which suggests that intermittent uplift was occurring during deposition of the sediments in both basins (Stearn et al. 1979).

The sedimentary rocks of Southern Ontario, within the basins, range from Cambrian to Devonian, with the younger formations outcropping at the surface in south-western Ontario. The sediments were derived from the Taconic Mountains (Figure 9.2c). The Queenston and Georgian Bay formation were deposited during the Upper Ordovician. The Queenston Formation gradationally overlies the Georgian Bay Formation.

9.3.1 Regional Character

The Queenston Formation outcrops along the base of the Niagara Escarpment, which runs from northern New York State, along the western shore of Lake Ontario and up to the tip of the Bruce Peninsula, where it continues below the water of Lake Huron (see Figure 9.1). It lies over the shales and interbedded limestones of the Georgian Bay Formation and is separated at its upper boundary by an unconformity with the Whirlpool sandstone, at Niagara Falls. The Whirlpool is present at the NTP site. The Whirlpool grades into the dolostones of the Manitoulin Formation.
(Winder and Sanford 1972). The Manitoulin Formation is only observed in the Michigan Basin. The Whirlpool is present in the Michigan Basin. However, it thins and ultimately disappears northwest of the Algonquin (Bergstrom et al. 2011).

On the regional scale, the formation can include sandstone and conglomerate near the erosional source on the east coast of North America to fossiliferous carbonates near Lake Huron (Tamulonis and Jordan 2009). Brogley et al. (1998) stated that the Queenston was deposited in a subtidal to supertidal depositional environment in Ontario, which changed to a fluvial-dominated environment in central New York and Pennsylvania. The Queenston Formation in Southern Ontario is predominately calcareous mudstone to red shale and can contain interbeds of siltstone and limestone.

The thickness of the Queenston Formation decreases in a north-westerly direction, from greater than 300 m at the NTP site to 73 m at the DGR site (Sandford 1961). As the Queenston thins, it grades into the upper part of the Georgian Bay Formation (Armstrong and Carter 2006). The regional variations in the depositional environment influence the site-specific strength, stiffness, and stress levels differently. However, similarities still exist despite the distance between the two sites.

9.3.2 Site Comparison
To numerically back analyse the NTP and evaluate the potential degree of excavation damage at the DGR, the site specific properties are compared to determine if they are within suitable ranges to make similar numerical methods applicable. To compare the laboratory testing results on intact Queenston core samples, the depth datum has been taken as the top of the Queenston Formation for each site. This is an imperfect datum as the Queenston thickness varies considerably between the two sites. However, it does give a frame of reference for comparison and should account for
the effects of regional changes in deposition on the properties. Greater local variations in deposition may account for the differences in the strength and stiffness trends between the two sites presented in this paper.

9.3.2.1 Numerical Model Inputs
The Unconfined Compressive Strength (UCS), Crack Damage (CD) and Crack Initiation (CI) values are important input parameters for brittle modelling, as defined by Diederichs and Martin (2010) according to the constitutive model of Diederichs (2007) and illustrated in Figure 9.4.

Figure 9.4: DISL spalling conceptual model (Diederichs 2007) with inset showing transition from lab testing CD threshold, for yield, to lower bound CI for spalling rocks. Other rocks yield in shear or show a combination (transitional) behaviour (from Perras et al. 2013).
For hard, brittle rocks such as granite, it is well known that the in situ strength drops from the yield threshold (CD) to a lower bound value (CI) determined during laboratory testing (Diederichs 2003). The reason for this drop is complex (Diederichs 2003), but it is particularly sensitive to confinement such that as the confining stress increases, the ability for cracks to propagate (reach CD) once initiated (at CI) becomes limited. Away from an excavation for example, it is possible to have micro-crack damage with no visible or significant mechanical influence (outer Excavation Damage Zone or EDZo), as discussed in Chapter 8. Closer to the excavation surface, cracks are less confined and more capable of propagating and connecting, which reduces the stiffness and ultimate strength of the material and increases the rock mass permeability. Near excavation fractures, once initiated, propagate spontaneously such that the observed wall strength of the excavation drops to the lower bound CI value. This model is applicable to crystalline rocks and is labelled as the ‘spalling rock mass’ curve in the inset of Figure 9.4.

Other rock types such as mudstones and siltstones do not necessarily follow this model. Cracks may not spontaneously propagate as they do in granites. Thus, damage may not develop into observable or significant mechanical damage as is observed in the inner Excavation Damage Zone (EDZi) or the Highly Damaged Zone (HDZ).

These rocks would behave as a ‘shearing rock mass’, as shown in the inset of Figure 9.4 (upper left). If, however, the plane of weakness (bedding plane) is parallel to the orientation of the most likely extension crack propagation direction, then the damage that begins at CI will migrate to these bedding planes and exploit them for propagation and ultimate failure. This behaviour can be considered transitional between shearing and spalling. In any case, the thresholds for CD and CI are important mechanical parameters for damage and failure prediction.
9.3.2.2 Mechanical Properties of the Queenston

Extensive testing for both projects has been carried out, including unconfined, triaxial, and tensile tests. These are the fundamental tests required to describe the failure envelope of the intact rock and the rock mass. The NTP testing was conducted at various laboratories over an extended period of time during the investigation stage of the project (the mid-1980s to 1998). During this time frame, it was established that the rock mass failure around excavations often occurred when the stress concentration exceeded 30%–50% of the peak laboratory strength, or the CI threshold. However, the importance of CI as an input parameter for numerical brittle spall prediction was not yet widely accepted in practice during the design of the Niagara Tunnel Project.

Although numerous UCS tests were conducted, as shown in Figure 9.5a, only a limited number of the completed tests included volumetric strain measurements, which can be used to determine CD and CI. The volumetric strain reversal point and the onset of the non-linearity of lateral strain points were used to determine the CD and CI thresholds, respectively, based on the test data from the NTP (courtesy of Ontario Power Generation). The values for the DGR were taken directly from various testing reports (Gorski et al. 2009, 2010, 2011). The reader is referred to Ghazvinian et al. (2013) for details on the methods for determining CD and CI. For clarity, only UCS and CI are plotted in Figure 9.5, with respect to the depth datum and percentage of siltstone content. The test results are also summarized in Table 9.1.

The strength values in Figure 9.5 have been plotted using the top of the Queenston Formation as the datum. At first inspection, it seems that there is a wide range of strength values for both the NTP and DGR. In fact, Figure 9.5a indicates a wider range at the DGR site. The DGR UCS and CI values increase with depth (Figure 9.5a and 9.5b).
Figure 9.5: Comparison of a) UCS, b) CI, and c) Young’s Modulus, $E_i$, between the NTP and the DGR of the Queenston Formation. Influence of the siltstone content is shown in d).

Table 9.1: Summary of properties for the Queenston Formation at the NTP site [2 and raw stress-strain data courtesy of Ontario Power Generation] and the DGR site as reported in Gorski et al. (2009, 2010, 2011)

<table>
<thead>
<tr>
<th></th>
<th>CI (MPa)</th>
<th>CD (MPa)</th>
<th>UCS (MPa)</th>
<th>T (MPa)</th>
<th>$mi$</th>
<th>$E_i$ (GPa)</th>
<th>$\nu$</th>
</tr>
</thead>
<tbody>
<tr>
<td>NTP</td>
<td>Avg.</td>
<td>15.3</td>
<td>27.5</td>
<td>39.0</td>
<td>2.48</td>
<td>11</td>
<td>11.3</td>
</tr>
<tr>
<td></td>
<td>Min.</td>
<td>8.1</td>
<td>14.9</td>
<td>15.4</td>
<td>1.09</td>
<td>5</td>
<td>5.4</td>
</tr>
<tr>
<td></td>
<td>Max.</td>
<td>42.4</td>
<td>112.1</td>
<td>112.9</td>
<td>4.29</td>
<td>14</td>
<td>32.2</td>
</tr>
<tr>
<td>DGR</td>
<td>Avg.</td>
<td>22.2</td>
<td>36.8</td>
<td>52.8</td>
<td>-</td>
<td>-</td>
<td>17.4</td>
</tr>
<tr>
<td></td>
<td>Min.</td>
<td>7.6</td>
<td>15.1</td>
<td>18.8</td>
<td>-</td>
<td>-</td>
<td>5.0</td>
</tr>
<tr>
<td></td>
<td>Max.</td>
<td>33.8</td>
<td>75.4</td>
<td>85.5</td>
<td>-</td>
<td>-</td>
<td>34.4</td>
</tr>
</tbody>
</table>
This could be partially related to the transition to the Georgian Bay Formation, at the base of the Queenston, which contains more siltstone and limestone interbeds. This also accounts for the increasing stiffness with depth at the DGR site (Figure 9.5c).

The thickness of the Queenston Formation is over 300 m at the NTP site, and only the upper portion was investigated for the tunnel. In Chapter 5 it was shown that at approximately 125 m below the top of the Queenston there is a large increase in UCS associated with the higher siltstone content at that horizon. This horizon was not considered for the back analysis because it is roughly 60 m below the low point of the tunnel. The UCS values in the upper portion of the Queenston Formation at the NTP site show a wide range, which is generally consistent with the depth. A closer examination indicates that there are potentially three strength bands, which all exhibit increasing strength with depth. The first band is at 0–25 m, whereas the second and third have depth ranges of 25–75 m and 75–100 m, respectively. Similar bands can be seen in the CI thresholds. These bands are likely related to changes in the depositional environment.

The carbonate content of the Queenston, including disseminated crystals in the shale matrix and interbeds of limestone, increases to the northwest away from the Taconic source zone, and it has been reported that the lower part of the Queenston consists of thinly interbedded and interlaminated siltstone, sandstone, and limestone, with red and green shale (Armstrong and Carter 2006). Thus, the increasing strength and stiffness with the depth and distance from the source could be associated with the increase in the calcite content.

A limited number of samples from the NTP were examined during the investigations for the project to determine the percentages of siltstone and shale within the sample tested. Examining the UCS values with respect to the siltstone percentage shows that the UCS generally
increases with increasing siltstone content, whereas the CI value is largely unaffected below 80% (Figure 9.5d).

Amann et al. (2011) investigated crack initiation in the Opalinus clayshale and indicated that tensile cracks began in the stiffer layers, and shear cracks began in the softer layers as a result of the stiffness contrast. In the case of the Queenston, the siltstone layers are stiffer and, according to Amman et al. (2011), should be where cracks first occur. Because CI has been determined as the point where the lateral strain deviates from linearity, the data suggest that the lateral strain deviation is controlled by the presence of the siltstone, irrespective of the percentage (up to 80%). The lateral stiffness is controlled by the shale layers (even at higher siltstone content), and because the stiffness of the siltstone is incompatible with the shale, tensile cracks develop in the siltstone. Above 80% siltstone, a sample’s lateral stiffness must switch to being controlled by the siltstone, which is stronger. The result is a high CI. The siltstone layers absorb cracks during loading, which cannot propagate through the shale layers. This influences the peak strength. With increasing siltstone, this peak strength also increases because there is a greater volume for crack absorption in the sample during loading. The layering also gives rise to anisotropic strength and stiffness, but does not influence CI, as mentioned previously.

The Direct Tensile Strength (DTS) was only measured on a limited number of samples for the NTP because of the difficulty in preparing such samples. The average DTS was determined to be 1.45 MPa for samples tested perpendicular to the bedding. This value can be considered to be the tensile strength of the bedding planes within the Queenston Formation at the NTP. The minimum DTS is reported in Table 9.1. Brazilian tensile strength (BTS) testing was more commonly completed for the NTP, and the average BTS, 4.29 MPa, was used as the maximum tensile strength because it has been determined that BTS is typically 30% higher than
the equivalent DTS for sedimentary rocks (Chapter 4). The same tensile values were used for the DGR models.

For the back analysis of the NTP, for tunnel chainages of 3000 to 3500 m, more specific UCS values were available. The minimum and maximum values are reported in Table 9.1. For both the NTP and DGR, three groups of properties were used as input for the numerical models. The minimum and maximum values, reported in Table 9.1, were considered to represent the range of values over six standard deviations. These were used to determine plus or minus one standard deviation and, along with the average values, these three groupings of properties (Table 9.2) were used in the numerical models to understand the influence on the overbreak dimensions.

9.3.2.3 Stress Conditions
Throughout southern Ontario, high residual in situ horizontal stresses exist in the sedimentary rocks, which were locked in as a result of tectonic activity during the Appalachian mountain building events, sedimentary basin effects and glacial loading and erosion. Stress shadows can occur at formation boundaries as a result of differences in the elastic properties (Haimson 1983).

### Table 9.2: Specific strength and stiffness values for the NTP (Perras et al. 2013) and for the DGR shaft (Gorski et al. 2009, 2010, 2011) used in the numerical modeling.

<table>
<thead>
<tr>
<th></th>
<th>CI (MPa)</th>
<th>CD (MPa)</th>
<th>UCS (MPa)</th>
<th>T (MPa)</th>
<th>Ei (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NTP</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>+1 St. Dev.</td>
<td>17.5</td>
<td>31.8</td>
<td>49.1</td>
<td>3.0</td>
<td>15.8</td>
</tr>
<tr>
<td>Mean</td>
<td>15.3</td>
<td>29.8</td>
<td>44.7</td>
<td>2.5</td>
<td>11.3</td>
</tr>
<tr>
<td>-1 St. Dev.</td>
<td>13.0</td>
<td>27.8</td>
<td>40.3</td>
<td>2.0</td>
<td>6.8</td>
</tr>
<tr>
<td>DGR</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>+1 St. Dev.</td>
<td>24.8</td>
<td>46.8</td>
<td>64.0</td>
<td>3.0</td>
<td>22.3</td>
</tr>
<tr>
<td>Mean</td>
<td>20.4</td>
<td>36.8</td>
<td>52.8</td>
<td>2.5</td>
<td>17.4</td>
</tr>
<tr>
<td>-1 St. Dev.</td>
<td>16.0</td>
<td>26.7</td>
<td>41.7</td>
<td>2.0</td>
<td>12.5</td>
</tr>
</tbody>
</table>
At the NTP, the deepest section of the tunnel, in the Queenston, is 140 m below the ground surface. For the purposes of the modelling in this paper, the vertical stress has been assumed to be the weight of the overlying rock mass. In Chapter 5 it was shown that there is a stress magnitude discontinuity at an elevation of approximately 40 m, which is approximately 6 m below the deepest section of the tunnel. This results in a wide maximum horizontal-to-vertical stress ratio ($K_0$) range of 2–9, and a horizontal stress ratio ($K_{Hh}$) range of 1–2.5 in the Queenston Formation, as shown in Figure 9.6. The typical $K_0$ at the elevation of the tunnel (40 m) ranges between two and six. The wide range of potential stresses has been used to determine the
variations in the numerical predictions of the depth of yielding in comparison to measurements from the NTP. Stresses at the DGR site have been estimated using a variety of methods (Intera 2011; NWMO 2011). Ko has a range of 0.5–1.6; KHh has a range of 1.0–3.2; and Khv has a range of 0.5–1.2 (NWMO 2011).

9.4 Overbreak at the NTP

Observations for the first 3.5 km of the tunnel were documented in Chapter 5 and defined four zones of behaviour (Figure 9.2b), three of which are within the Queenston. Zone 1 is defined as all the formations above the Queenston. Zone 2 is at the contact between the Whirlpool and Queenston formations, which is a disconformity. The reduction in stress due to a stress shadow, and jointing, created conditions permitting large blocks to fall from the crown. The overbreak was observed to break back to the overlying Whirlpool Formation to a maximum depth of 1.4 m, at which time forward spiling support was used to advance the tunnel. When the tunnel reached its maximum depth (140 m), stress-induced failure was observed. However, the behaviour was influenced by St. Davids Buried Gorge, which the tunnel had to pass under.

On reaching the structural influence of the buried gorge (Zone 3), the overbreak was on the order of 2.0 m, as shown in Figure 9.7. It should be noted that through most of this zone, forepoles were used. Vertical jointing, spaced 2-3 m, and horizontal and inclined shear surfaces were observed. The joints remained clamped as a result of the stress concentration and had a minor influence on the overbreak geometry. The shear surfaces likely affected the overbreak, although this was not observed. The overbreak geometry remained asymmetric throughout this zone. However, it was generally inconsistent in size and shape because of the irregular depth to the bottom of St. Davids Buried Gorge.
Stress induced fracturing became more prominent as the tunnel moved away from the influence of the buried gorge, marking the transition to stress-induced overbreak in Zone 4. The crown overbreak formed an arch 7–8 m wide with a consistent notch shape, skewed to the left as shown in Figure 9.8, which likely indicates a high stress ratio with the major principal stress orientation slightly inclined from the horizontal. The overbreak reached a maximum depth of 6 m. However, it was more typically in the 3–4 m depth range.

Failure in the invert continued with induced spall planes, which were marked with plumose and conchoidal surfaces. Although minor sidewall spalling occasionally occurred in the sidewall area, it was limited to vary shallow surficial damage when it occurred.
Figure 9.8: Typical overbreak profile for the high horizontal stress field in overbreak Zone 4. Inset photo showing overbreak up to ~ 3 m deep (from Perras 2009).

Detailed measurements of the crown maximum overbreak, apex angle, and chord closure were presented in Chapter 5, up to a chainage of 3500 m within the tunnel. The maximum, mean, and minimum overbreak and the chord closure measurements between tunnel chainages of 3000 and 3500 m have been used for the back analysis.

This section marks the transition from the influence zone of St. Davids Buried Gorge into the higher stress field. The numerical modelling has focused on the crown measurements to determine the strength and stress conditions that give rise to similar numerical results. With an understanding of the appropriate modelling method, the input properties that gave reasonable results similar to the observations at NTP have been used for forward numerical prediction at the DGR.
9.5 Numerical Models

The notch at the NTP in the Queenston Formation formed through brittle failure, as observed by the authors. The brittle failure process can be captured numerically using the Damage Initiation and Spalling Limit (DISL) approach of Diederichs (2007). Work by Perras (2009) demonstrated that when the anisotropic stiffness is captured in a numerical model using joint elements, the overbreak geometry can be correctly simulated. These approaches were implemented in the finite difference program Phase 2, by RocScience, as a preliminary assessment to determine the appropriate stress and rock mass properties for a more advanced analysis. To capture the influence of horizontal laminations on a vertical shaft, three-dimensional numerical models are necessary. Advanced analyses have been conducted using the finite difference program FLAC 3D, by Itasca, to capture the lamination influence on the rock mass behaviour. This has been done using the ubiquitous joint double yield (UBJT) model in FLAC 3D (Itasca 2009). The modelling methods will be discussed in more detail below.

9.5.1 Failure Criteria

Brittle failure is the result of extensile fractures forming parallel to the excavation surface under compressive loading. A focal point of stress creates localized damage, which then concentrates the stress around the local damage. This in turn creates more localized damage. In this manner, the damage is localized into a notch geometry, which is typical for brittle failure around underground excavations (Martin 1997).

Several numerical approaches have been used to capture the brittle behaviour process (Martin 1997; Hajiabdolmajid et al. 2002; Diederichs 2007). Diederichs (2007) developed a method to represent brittle behaviour using the generalized Hoek–Brown (Hoek et al. 2002) peak and residual parameters, which are standard input parameters for engineering design software.
The DISL method of Diederichs requires CI, the UCS thresholds, and the tensile strength as input properties. Rocks with UCS / T > 9 and rock masses with GSI > 55 can behave in a brittle manner (please note the specific limits in Diederichs 2007). Using the Generalized Hoek–Brown (Hoek et al. 2002) parameters the peak and residual failure curves can be determined, using Equations 9.1 and 9.2, after Diederichs (2007):

\[
s_p = \left( \frac{CI}{UCS} \right)^\left( \frac{1}{a_p} \right) \tag{9.1}
\]

\[
m_p = s_p \left( \frac{UCS}{|T|} \right) \tag{9.2}
\]

where \(a_p\) is a curve fitting parameter for the peak curve, taken as 0.25 in this paper. Diederichs (2007) suggested that the residual parameters, \(a_r\), and \(s_r\), should be 0.75 and 0.001, respectively. The residual parameter \(m_r\) should be between six and ten, and six was used in this paper for the DISL approach. The mean peak and residual DISL failure envelopes for the NTP and DGR are shown in Figures 9.9a and 9.9b, respectively.

Ghazvinian et al. (2013) indicated that the Queenston Formation has anisotropic stiffness and strength. Perras (2009) demonstrated that the anisotropic behaviour could be simulated using joint elements to capture the anisotropic stiffness of the rock mass. To ensure compatibility between the laminated area (with joints) and the non-laminated area, a relationship for the transversely isotropic elasticity was used to scale the modulus, which accounted for the normal stiffness \((K_N)\) and spacing of the laminations \((T)\) (see Equation 9.3). Using these parameters and the beam modulus (in this case the intact modulus), \(E_{\text{beam}}\), a non-laminated rock mass modulus, \(E_{\text{rm}}\), can be equated using Equation (9.3), as shown below (Brady & Brown 2006).
\[
\frac{1}{E_{rm}} = \frac{1}{E_{beam}} + \frac{1}{K_NT}
\]  
(9.3)

The rock mass (without laminations) and the rock beams (in between laminations) have been modelled as a perfectly plastic Hoek–Brown (Hoek et al. 2002) material. The Hoek–Brown (Hoek et al. 2002) parameters, \(m_b\) and \(s\), were also scaled, such that the rock mass properties were compatible with the rock beams. This was done first by adjusting the GSI value, such that the modulus was the same as that calculated using Equation 9.3, and then taking the \(m_b\) and \(s\) values and harmonically averaging these with the beam \(m_b\) and \(s\) values, following the methodology of Perras (2009). The intact, rock mass, and beam failure envelopes are shown in Figure 9.9c.

The laminations provide a surface for lateral slip and detachment during convergence and deflection, respectively, which is not accounted for when using isotropic models such as the DISL approach. By modelling horizontal laminations with joint elements, the rock mass behaviour is controlled by both the beams and the laminations themselves.

The laminations reduce the rock mass modulus in the vertical direction and allow for greater joint parallel displacements over an equivalent isotropic numerical representation of a rock mass (Perras 2009). The laminations also allow for deflection of the rock beams into the excavation. The increases in the joint parallel displacement and beam deflection create a different deformed excavation boundary surface for the laminated model compared to the equivalent isotropic model. In the modelling presented in this paper, three different joint element Mohr–Coulomb failure envelopes were used to define the lamination properties, as shown in Figure 9.9d. Perras (2009) showed that this numerical approach creates a plastic yield zone similar to that observed at the NTP.
Figure 9.9: The failure envelopes for: a) the NTP – DISL and UBJ models, b) the DGR – DISL and UBJ models, c) the NTP – the laminated (lam.) models, and d) the lamination Mohr-Coulomb envelopes.
The UBJT model allows for two Mohr–Coulomb segments to be used to define the failure envelope, as well as a tension cut-off. This model was chosen because of the simplicity of the input parameters, which only require cohesion, friction, and tensile cut-off values. Residual properties are activated by plastic strain levels over user-defined stages and do not require other plastic indicators to control the transition to residual properties, such as are required for the implementation of the DISL approach with the generalized Hoek–Brown (Hoek et al. 2002) failure criterion in FLAC3D (Itasca 2009). The model can consider a weaker plane of anisotropy. For this study, three different property sets were evaluated (Figure 9.9d and Table 9.2).

The brittle rock mass properties were implemented in the UBJT model by selecting envelopes that approximate the DISL peak and residual envelopes. This was done by projecting the first segment of the yield surface from the tensile (T) strength to CD (see Figures 9.9a and 9.9b).

This appears to adequately capture the curvature of the DISL peak yield surface for both the NTP and the DGR. The second segment of the peak UBJT yield surface is fit between CI and the intersection of the DISL peak and residual envelopes. The residual DISL curve is approximated manually using a tensile strength close to zero.

The UBJT model allows for peak and residual properties to be captured using a strain softening/hardening approach, by utilizing the plastic shear strain as an indicator of when to decrease/increase the property. The properties used in the numerical modelling are summarized in Table 9.2. The plastic shear strain increments used to control the transition from the peak to residual in the FLAC3D models were determined following the work of Hajiabdolmajid (2001).
9.5.2 Geometry

For the NTP models, a back analysis was first conducted in two dimensions to determine the ranges of the strength and stress values that capture the observed overbreak geometry, as shown in Figures 9.7 and 9.8. Two-dimensional cross sections were modelled at tunnel chainages of 3000, 3250, and 3500 m, which corresponded to tunnel invert elevations of 46, 47, and 61 m, respectively. Three-dimensional models were also used to make a comparison with the laminated approach.

A radial mesh was used for the modelling, with an outer boundary radius of 70 m (5x the tunnel diameter). The curved outer boundary is fixed in both the vertical and horizontal directions, and in the three-dimensional case, the model can move out of the plane, parallel to the tunnel orientation, with the ends of the models fixed. For the NTP models, the interior region near the excavation surface has zones with lengths of 0.16 m and 0.5 m for the two- (DISL) and three-dimensional models, respectively. The laminated models capture the true stratigraphy of the NTP because of the close proximity of the contact between the Whirlpool and the Queenston. In this case, a rectangular model boundary is used, with dimensions similar to the radial mesh. The mesh, however, is constrained by joint elements spaced 0.2 m apart. In any case, the zones gradually begin to increase in size away from the excavation surface (Figure 9.10). The tunnel excavation was completed in 2-m stages over a length of 75 m.

A similar setup was used for the DGR models (Figure 9.10). The zones at the excavation surface of the DGR shaft models have lengths of 0.06 and 0.45 m for the two- and three-dimensional models, respectively. The two-dimensional models have a much finer mesh because they have been used in Chapter 8 to study of the development of the EDZ and a desired scale of 0.1 m was used.
Figure 9.10: Mesh setup for the NTP (top) and the DGR (bottom) showing zone dimensions (see insets for detail) increasing away from the excavation surface. The axes arms are 5.0 m.
9.5.3 Rock Support Considerations
Overbreak measurements from the NTP show that once the tunnel had passed under St. Davids Buried Gorge and into the high regional stress field, the depth of the overbreak was typically greater than 2 m. The depth increased to the order of 4 m around a chainage of 3500 m and reportedly reached a maximum of 6 m beyond 3500 m. A gap of roughly 6 m between the face and the point of the primary rock support installation forced the removal of the yielded rock mass prior to the installation of bolts and steel channels. The notch dimension were measured prior to support installation and minimal visible deformation was observable after support was installed. Forward spiles were used to bridge the gap to minimize the volume of overbreak being removed from the tunnel crown. In areas where spiles were not installed, the notch could fully develop, and these areas are of primary interest for the back analysis and support was not considered.

9.6 Model Results
The NTP provides an opportunity to back analyse the numerical stress and strength scenarios that most closely match the measured overbreak and chord closure measurements. The comparison between DISL and the laminated models is used to demonstrate the need to capture the anisotropic strength and stiffness to correctly capture both the overbreak dimensions and the chord closure measurements. In two dimensions, the influence of horizontal lamination cannot be captured for the DGR site models because the lamination plane is in the same plane as the numerical model. In this case, the DISL and UBJT models are compared.

9.6.1 Back Analysis of the NTP
An example of the typical notch that formed when spiles were not installed is shown in Figure 9.11, and measures 3.78 m deep. Because the notch was fully formed prior to the installation of the rock support, when spiles were not installed, the numerical simulation of the rock support has
been neglected. Thus, the numerical results should yield the maximum notch geometries. The observed depth of the overbreak from Figure 9.11 was used as a target to determine the starting stress state for the analysis using the empirical relationship of Martin et al. (1999), which was modified to include CI for brittle spall modelling by Diederichs (2007).

Figure 9.11: Typical overbreak notch at the Niagara Tunnel Project encountered in the high horizontal stress zone, Zone 4 from Figure 9.2b.
The DISL or the similar cohesion weakening friction hardening (CWFH) modelling approaches have been shown to be very effective in capturing the correct notch geometry associated with brittle rock mass failure (Diederichs 2007; Hajiabdolmajid et al. 2002; Hajiabdolmajid 2001).

Back analysis was conducted at three different tunnel chainages; 3+000, 3+250, and 3+500. These chainages were selected because the overbreak depth changed from 2 to 4 m over this 500 m section of the tunnel. The back analysis modelling was conducted in stages, with a layer of complexity added at each stage to narrow down the range of inputs that correctly capture the overbreak and chord closure measurements. The stages that were used are as follows:

a) DISL models with mean properties over a wide range of stress scenarios
b) DISL models with ±1 standard deviation of the mean properties
c) DISL models including dilation
d) LAM models with mean properties over a narrowed stress scenario range
e) LAM models with varying joint element properties
f) UBJT models with mean properties.

The starting Ko ratio was determined using the mean empirical depth of the relationship presented by Diederichs (2007):

\[
\frac{r}{R} = 0.5 \left( \frac{\sigma_{\text{max}}}{CI} + 1 \right)
\]  

(9.4)

where \(r\) is the maximum depth of the notch, \(R\) is the radius of the excavation, and \(\sigma_{\text{max}}\) is the maximum tangential stress at the excavation boundary calculated by \(3\sigma_1 - \sigma_3\).

The target depth of failure was taken as 4 m (chainage 3500 m), and the radius of the tunnel was 7.2 m. Solving Equation 9.4 by using the average CI value of 15 MPa, the maximum tangential stress at the boundary \(\sigma_{\text{max}} = 30\) MPa. The maximum tangential stress for the NTP models was determined using \(\sigma_{\text{max}} = 3\sigma_H - \sigma_v\). A vertical stress gradient of 0.026 MPa/m was assumed, and therefore at 140 m below the ground surface, \(\sigma_v = 3.64\) MPa. Solving for \(\sigma_H\), the
maximum horizontal stress would be 11 MPa. This results in a Ko ratio of 3.4. For simplicity, 3.5 was used as the starting point.

Perras et al. (2013) found that the typical 4-m notch geometry could be captured using the mean rock properties and a Ko of 4 using the DISL approach. However, using these same values and changing the elevation of the model sections does not adequately capture the measured depth of the overbreak. To capture the changing overbreak depth at different chainages, the stress field was modified, and the mean rock properties were adjusted by ±1 standard deviation.

Diederichs (2007) stated that the DISL method on its own does not adequately capture displacements, but dilation should be used to induce reasonable displacements. Generally, a higher dilation angle allows for a greater post-yield volumetric expansion of the rock mass. This results in increased strains, over zero dilation models, around the modelled excavation in the plastic yield zone and can be used to correctly capture the strains and displacements. Vermeer and de Borst (1984) recommended using a dilation angle, Ψ, that was smaller than the friction angle, φ. Hoek and Brown (1997) stated that values for Ψ are typically around φ/8. These are equivalent to the dilation parameter m_d and m_r, respectively. Walton et al. (in press) suggested estimating the appropriate constant dilation angle using Equation 9.5:

\[
\frac{\Psi}{\phi} = 0.5 \left( \frac{C_l}{\sigma_{max}} \right) - 0.1
\]

The results of the two-dimensional DISL models are plotted against the measured overbreak and chord closure limits in Figures 9.12a and 9.12b, respectively. Due to the constraint of the TBM head, chord closure measurements at the NTP could only begin to be measured approximately 6 – 7 m back from the face. In order to correct the closure measurement limits the elastic convergence, \(u_{ie}\), can be estimated using Equation (9.6):

\[
u_{ie} = \frac{r_o(1+v)}{E} (p_o - p_i)
\]
where \( r \) is the tunnel radius, \( v \) is Poisson`s ratio, \( E \) is Young`s Modulus, \( p_o \) is the hydrostatic in-situ stress, and \( p_i \) is the internal support pressure. If the average Young`s Modulus and Poisson`s ratio (0.35) are used with the stress at 140 m depth in Equation 9.6, the resulting elastic convergence is 14 mm including convergence in front of the face.

According to Vlachopoulos and Diederichs (2009) the Longitudinal Displacement Profile (LDP), using a plastic radius of 11 m, would predict 25% of the maximum displacement to occur at the face. Yield at the face was not a common occurrence at the NTP. Therefore a conservative estimated correction of 25% of the elastic convergence or 3.5 mm has been applied to the limits of Figure 9.12b.

A variety of stress and strength inputs can yield overbreak dimensions, estimated from the maximum yield strain contour of 0.001, within the limits of the measured values from the NTP. To narrow the stress and strength scenario which yields the measured overbreak dimensions, at each chainage, dilation was used to increase the numerical chord closure to match the measured in-situ values. The results include models with a dilation of 0 and those with a range of dilation parameters between 0.1 and 6. Figure 9.12b shows that chord closures from the numerical results could only be captured at chainages of 3250 and 3500 m within the observed limits. At a chainage of 3000 m, the minimum achievable chord closure is shown in Figure 9.12b. In fact, when the dilation parameter is used, the chord closure first decreases as the dilation parameter is increased. However, a minimum chord closure is reached, and further increases in the dilation parameter increase the numerical chord closure again (Figure 9.13). Using dilation also increases the overbreak depth above the value of the same model without dilation. For the example shown in Figure 9.13, the models with dilation show only a marginal increase in the overbreak (0.1 to 0.2 m) over the model without dilation.
Figure 9.12: DISL and LAM model results in comparison to the measured a) overbreak and b) chord closure from the NTP (limits from Perras et al. 2013) with inset model examples.
Because the DISL approach with different dilation values is unable to produce chord closures below a certain level, the laminated (LAM) modelling approach was employed to control the lateral closure using the lamination properties. The LAM approach gives overbreak depth results similar to the DISL approach and can correctly capture the overbreak geometry (notch shape) within the measured limits (Figure 9.12a). The inclusion of the laminations induces anisotropic stiffness in the model, which is controlled by the joint element properties, as previously discussed.

Figure 9.13: Influence of the dilation parameter on the overbreak depth and chord closure determined from a DISL model at a tunnel station of 3250m using the +1 standard deviation properties and a $K_o = 3$ and a $KHh = 1$. 

400
The normal \((K_n)\) and shear \((K_s)\) stiffnesses of the intact bedding in a rock mass are difficult to measure accurately in the laboratory and are seldom reported in the literature. Savilahti et al. (1990) reported that for intact rock \(K_n = K_s\), and Barton (2007) stated that for very good joint surfaces, \(K_n/K_s\) should be between 11 and 15. For the modelling presented in this paper, \(K_n/K_s\) was tested between 1 and 11 because the bedding planes were intact. The shear stiffnesses were calculated using this range of ratios after the normal stiffness was determined using Equation 9.3.

Figure 9.14 shows the relationships between the stiffness ratio \((K_n/K_s)\) and the overbreak depth and chord closure results from an example model at a chainage of 3000 m, with mean rock properties and stress ratios of \(K_o = 2.5\) and \(KHh = 1.5\). The models demonstrate that there is a direct relationship between the stiffness ratio and the chord closure. As the stiffness ratio is decreased, the chord closure also decreases.

There is a less clear relationship between the overbreak depth and the stiffness ratio, although generally the overbreak depth increases with increases in the stiffness ratio. The more erratic relationship is due, in part, to stress channeling, which has been described in more detail by Perras (2009). The stress is channeled through the beams above the crown of the excavation, and each consecutive beam above the crown can build stress before completely yielding, including the failure of the joint element, which sheds the stress to the beam above. This creates a non-linear relation: as \(K_s\) is reduced, there is an increase in the horizontal displacement, which causes more convergence into the excavation (Perras 2009).

The target chord closure at a chainage of 3000 m has a range of 6–14 mm, and the target overbreak range is 0.9–3.2 m. The model results shown in Figure 9.14 indicate that a stiffness
ratio of less than three is needed to achieve the targeted chord closure and overbreak depth. The laminations in the model help to capture both the chord closure and overbreak targets.

However, the horizontal nature of the laminations in the Queenston Formation means that two-dimensional models can only capture this behaviour when a horizontal tunnel is modelled. For a vertical shaft, a three dimensional model is required to incorporate the anisotropic behaviour. As discussed previously, this has been done using the UBJT approach, which was first applied to the NTP to determine if it matched the results of the DISL and LAM approaches.

![Graph showing the influence of joint stiffness ratio on overbreak depth and chord closure](image)

**Figure 9.14**: Influence of the joint stiffness ratio, $K_n/K_s$, on the overbreak depth and chord closure determined from a LAM model at a tunnel station of 3000m using the +1 standard deviation properties and a $K_o=2.5$ and a $KH_h = 1.5$. 
The range of overbreak depths is illustrated in Figure 9.15 from the modelled results for the NTP. The maximum shear strain contours are shown on the left, with the plastic yielding on the right. The maximum shear strain contours give a more representative shape to the notch geometry, and visible damage can be expected to occur within the continuous zone of contours that intersect the excavation surface.

Utilizing the maximum shear strain contour approach, the model that most closely represents the NTP notch is shown in Figure 9.15 (bottom). The depth of the notch in the model is 3.85 m, and it is roughly 7.0 m wide at the tunnel crown, measured horizontally, similar to that observed at around 3500 m (Figure 9.11). The UBJ model has been shown to capture the behaviour for the NTP, and it incorporates ubiquitous joints that can capture the strength anisotropy of the Queenston Formation.

9.6.2 Rock mass Anisotropy and Excavation Orientation

As previously mentioned, to model the DGR shaft in the horizontally laminated Queenston Formation and incorporate the anisotropic strength, the UBJT approach has been applied. However, the appropriate properties for the rock mass should first be discussed.

As expected, the Queenston behaves in the typical anisotropic manner, with minimum strength values when the bedding is inclined at 45° to the loading axis, as shown in Figure 9.16. CD likely follows the same trend, although there were no stress–strain curves for the 45° samples available. The CI thresholds, however, are similar at both 0° and 90° and are in fact roughly in the same range as the peak strength of the 45° samples. If this behaviour at the laboratory scale is applied conceptually to the rock mass strength envelope, then the orientation of the excavation with respect to the orientation of the bedding planes changes the observed behaviour, as conceptually illustrated in Figure 9.17.
Figure 9.15: Comparison of maximum shear strain (left) and plastic yield (right) for the range of stress conditions modeled. $K_0=3$ (top) and $K_0=4$ (bottom).
Figure 9.16: UCS, CI and CD thresholds estimated by the strain method versus lamination angle for the Queenston Formation with average values indicated with the line (modified from Ghazvinian et al. 2013).

A UCS test with horizontal bedding should reflect the strength of the sidewall in a horizontal excavation with $\sigma_1$ orientated parallel to the horizontal bedding. Similarly, a UCS test
with vertical bedding should reflect the strength of the crown and invert in a horizontal excavation with \( \sigma_1 \) orientated parallel to the horizontal bedding. In a horizontal excavation in a rock mass with horizontal bedding, the beds in the crown and invert are able to deflect and fail into the excavation. Micro-cracks should propagate more easily along the bedding than across it.

Conceptually (Figure 9.17), this means that for a horizontal tunnel in horizontal bedding and high horizontal stress \( CD = CI \), and the rock mass will behave in a brittle fashion. For a vertical shaft in a rock mass with horizontal bedding, the beds are unable to deflect into the excavation, and this confinement allows for friction to be mobilized on the bedding planes if yielding occurs, which conceptually means \( CD > CI \) (Figure 9.17). The rock mass therefore would behave as a shearing rock mass. In addition, because a cross section through the vertical shaft in the rock mass with horizontal bedding would be parallel to the plane of anisotropy, the stiffness is unaffected by the bedding. In this case, there should be no advantage in modelling the anisotropic stiffness with a three-dimensional model.

### 9.6.3 Forward Prediction of the DGR shaft

Numerical modelling of the shaft through the Queenston Formation was conducted by NWMO (2011) using a variety of methods. The depth of the EDZ from NWMO’s study (2011) had a range of 2.03–3.42 m, as shown in Figure 9.18. The lower end of the range was predicted using the DISL approach, and the upper end was predicted using a strain weakening approach. Experience from the NTP would suggest that the behaviour at the tunnel was brittle in nature.

Numerical modelling of the shaft through the Queenston Formation was conducted by NWMO (2011) using a variety of methods. The depth of the EDZ from NWMO’s (2011) study had a range of 2.03–3.42 m, as shown in Figure 9.18. The lower end of the range was predicted
using the DISL approach, and the upper end was predicted using a strain weakening approach.

Experience from the NTP would suggest that the behaviour at the tunnel was brittle in nature.

In this study, two-dimensional modelling of the DGR shaft was also conducted using the DISL approach and the three rock mass property sets from Table 9.2. At the DGR site, the dimensions of the EDZ are of interest because of the potential flow path through this zone of damage. Engineered cut-off structures must be designed to intersect the EDZ to minimize the flow along the potential pathway.

![Figure 9.17: Interpretation of the DISL model combined with post CI interaction with bedding weakness planes in a tunnel and shaft.](image)
Figure 9.18: Maximum depth of plastic yield for sites. Solid and dotted lines are average, maximum and minimum spalling limits, respectively, based on Martin (1997).

To determine the range of the expected depth of damage, the stress ratios $K_a$ and $K_{Hh}$ were varied such that the normalized tangential wall stress ($\frac{\sigma_{max}}{CI}$) range fell between 1 and 2.7. Below a normalized tangential wall stress of one, there should be negligible damage around the excavation (i.e. isolated micro-cracking only), and the upper limit of 2.7 represents the maximum value based on the probable stress scenario at the DGR site within the Queenston Formation using the average CI (NWMO 2011).
The DISL results from this study are bracketed by the results presented by NWMO (2011), as shown in Figure 9.18. Even when using the –1 standard deviation properties, the results from this study are only approximately 0.15 m deeper than the maximum predicted depth of damage from the NWMO (2011) study. As expected, generally the models with high strength (+1 standard deviation) had lower depths of damage than the weaker models tested.

However, a consistent non-linear relationship between \( \sigma_{\text{max}} / \text{CI} \) and the depth of damage is demonstrated (Figure 9.18). When the \( \sigma_{\text{max}} / \text{CI} \) ratio is greater than two, the linear empirical limits tend to overestimate the depth of damage, when compared to all the numerical results.

Examining the UBJT results shows that when \( \sigma_{\text{max}} / \text{CI} \) is smaller than two, the model predicts less damage than the equivalent DISL model. Above this level, there is good agreement with the DISL model results. This suggests that the failure mechanism in the DGR case can be adequately captured with an isotropic continuum approach, and that the ubiquitous joints have minimal influence because of their orientation relative to the excavation.

Three examples are presented in Figure 9.19, which show the maximum shear strain contours and plastic yield around the excavation models. The top model shows variable plastic yield along the shaft. The plastic yield is associated with the staging of the excavation and the corner of the sidewall with the face. The average maximum depth was taken from the analysis.

The middle and bottom models in Figure 9.19 represent the intermediate and maximum stress scenarios, respectively. The latter shows a maximum depth of damage of roughly 2 m. Figure 9.19 also demonstrates that the shape of the damaged zone is different than that predicted by the NTP models. The notch is more rounded and wider in comparison to the NTP models. This is the result of the gravitational influence at the NTP and the larger stress ratio in the plane perpendicular to the excavation orientation.
Figure 9.19: Maximum shear strain (left) and plastic yield (right) for the minimum (top), intermediate (middle) and maximum (bottom) stress scenarios for the DGR shaft.
For the NTP, the in-plane stress ratio is $K_o$, with a maximum value of approximately five, whereas at the DGR, it is $KH_h$, with a maximum value of approximately 3.2. The possibility of a notch developing decreases as the in-plane stress ratio decreases. This is an important aspect to consider for such cases where cut-offs are required to minimize the flow of radionuclides through the inner EDZ.

9.7 Discussion
The notch at the NTP was influenced by the requirements to remove loose rock above the TBM shield prior to the installation of the rock support. Intense scaling was conducted and as each damaged bedding slab was removed during the scaling operations, the small amount of confinement provided by the damaged bedding slab was also removed. This allowed the damage to propagate deeper into the rock mass as scaling continued. This was similar to a gravel surcharge which may be used to minimize damage in the invert of a tunnel. Once the gravel is removed, the damage can occur (Diederichs 2007). In the case of the NTP, the process was assisted by gravity, but it was initiated by the sub-horizontal bedding orientation and stress concentration at the crown (Chapter 5).

Numerically, the plastic yield limit marks the extent of micro-cracking, and in the context of nuclear waste storage, micro-cracking does not mean increased permeability if the cracks do not coalesce. A methodology described by Perras et al. (2012) to determine the excavation damage zones from continuum models, demonstrated that the volumetric strain could be used to distinguish between different types of excavation damage, micro- and macro-cracks. These are interconnected, which increases the permeability (HDZ/EDZi), and isolated micro-cracks where the permeability changes do not occur (EDZo) within numerical models.
This is illustrated in Figure 9.20, for both the NTP and DGR cases. The maximum shear strain is plotted on the left, and the volumetric strain contours are plotted on the right. Within the zone of damage, there are both positive and negative volumetric strains. In the logic of FLAC3D, the contraction is negative, and the extension is positive (Itasca 2009). It can be seen in Figure 9.20 that the switch from contraction to extension occurs within the notch for both projects.

It should be noted that it is not the reversal point of the gradient, but in fact the sign change that marks the beginning of extension because there is an elastic volumetric strain that has to be overcome before extension can occur, as discussed by Perras et al. (2012). The EDZi at the NTP would be predicted numerically to be 1.5 m; however, as a result of the scaling and the near horizontal bedding, the overbreak propagated to the extent of the micro-cracking (EDZo), roughly 4 m in the calibrated model. Thus, in a shaft scenario, the overbreak should be less than the EDZo because gravity does not influence the damaged rock mass in the same manner as it does in a tunnel excavation. In the case of a shaft, the likely depth of scaling that could be achieved without excessive effort is likely to be the EDZi. In Figure 9.20, the difference between the dimensions of EDZo and EDZi is approximately 0.8 m.

To determine the relationship between the different modelling approaches for the NTP in a manner similar to that for the DGR (Figure 9.18), the results are normalized and plotted in Figure 9.21. In this figure, the model results that conformed to the measured overbreak depth and chord closure are plotted. For the case of the NTP, the UBJT model results show the highest sustained stresses that yield overbreak depths in the maximum target ranges. However, using joint properties with 80% of the rock mass strength, overestimated the chord closure when the mean rock mass properties were used, and underestimated it when the +1 standard deviation properties were used.
Figure 9.20: Maximum shear strain (left) and volumetric strain (right) for the NTP model with $K_o = 4$ and $KHh = 1.4$ (top), the best fit UBJT model, and for the DGR model with $K_o = 1.6$ and $KHh = 3.2$ (bottom), which represents the maximum depth from the DGR models. The axis is 4.05 m in length.
The LAM models are also able to capture both the maximum overbreak depth ranges and the chord closure, however, only at reduced confining stresses over the UBJT models. None of the models were able to correctly capture both the overbreak depth and chord closure at chainage 3500 m. It was possible to capture one or the other using the variations in the properties. It is possible that the majority of the overbreak at 3500 m should be close to 4 m in depth, and that, on occasion; depths of 6 m may have been encountered as a result of other geological structures or by excessive scaling.

Figure 9.21: Normalized tangential wall stress versus overbreak depth for the numerical models that matched the measured overbreak and closure measurements.
The plastic yield and maximum shear strain contours show the typical notch-shaped geometry observed in other excavations in brittle rocks, such as at the underground research laboratory (URL) operated by Atomic Energy of Canada Ltd. (AECL) for example (Martin 1997). As the stress ratio increases, the plastic yield zone becomes less of a notch and ‘stringers’ of plastic yielding begin to occur when the stress ratio increases beyond the slope of the Mohr Coulomb failure envelope in the $\sigma_1-\sigma_3$ space. These ‘stringers’ are in fact realistic damage that represents isolated micro-cracks that do not coalesce into visible damage because they remain in a confined state in the rock mass. This has been demonstrated at AECL’s URL by monitoring the micro-seismic activity in front of an excavation face (Martin 1997).

The target mean normalized damage radii, $r/R$, are indicated for the different chainages in Figure 9.21, and it can be seen that the empirical limits underestimate the required normalized wall stress predicted by the models, similar to the DGR models. The DISL and LAM model results in Figure 9.21 were able to capture both the overbreak and chord closure limits that were measured at each chainage. The UBJT models were unable to capture both target measurements.

The corresponding ranges of stresses and properties that result in overbreak and closure results similar to those measured at the NTP are shown in Table 9.3. By matching both the overbreak depth and chord closure measurements with the numerical results there is increased confidence, over only match one criteria, that a unique solution has been determined. These results generally indicated that the Ko ratio increases with depth, and the strength also increases with depth, which is consistent with the measurements presented earlier (Figures 9.5 and 9.6).

The in-situ variability of the strength, stiffness, and stress and the installation of rock support during excavation accounts for the range of the measured overbreak depths and chord closure values presented in Chapter 5.
Table 9.3: Final ranges of stress ratios and properties sets at the three chainages modeled which gave rise to overbreak depths and chord closures within the measured limits.

<table>
<thead>
<tr>
<th></th>
<th>3000 m</th>
<th></th>
<th>3250 m</th>
<th></th>
<th>3500 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_o$</td>
<td>2.5-4.0</td>
<td>+1 St. Dev.</td>
<td>3.5</td>
<td>+1 St. Dev.</td>
<td>3.2</td>
</tr>
<tr>
<td>$K_{Hh}$</td>
<td>1.2-1.5</td>
<td></td>
<td>2.9-3.6</td>
<td></td>
<td>1.25</td>
</tr>
<tr>
<td>Properties</td>
<td>+1 St. Dev.</td>
<td>Mean</td>
<td>1.0-1.8</td>
<td>Mean</td>
<td>Mean</td>
</tr>
</tbody>
</table>

The rock support would only have a small influence on the depth of yielding because of the delay in installation. However, the yielded material is retained, and the true depth of yielding is un-measureable. Confinement is provided by the rock support and could be sufficient to suppress further propagation of the notch when it is beyond EDZo in-situ.

This study was aimed at a back analysis of the NTP overbreak and closure. The different modelling approaches were calibrated to the measurements from the project. The understanding gained from this back analysis was applied to the DGR site to investigate the performance of the shaft in this formation at a deeper level but with a much less severe stress ratio between any of the principal stresses.

9.8 Conclusions

The back analysis of the NTP showed the importance of incorporating the anisotropic stiffness and strength because the LAM modelling approach was the most successful at capturing the measured overbreak depth and chord closure measurements from the site. All three modelling methods were able to capture the measured overbreak geometry and chord closure. However, the maximum projected targeted values at a chainage of 3500 m could not be captured using the same model properties. Individually, the overbreak geometry or the chord closure measurements could be captured with all the modelling approaches. For the models that captured both target values, a
narrower range of \( K_o \) and \( K_{Hh} \) at the specific chainages was determined over the general ranges stated for the whole project.

A similar methodology was implemented to predict the depth of damage around the DGR shaft in the Queenston Formation. The results were in agreement with those of previous studies. All of the modelling methods gave similar depths of damage above a \( \sigma_{CI} \) value of two. Below this threshold, conservative maximum damage depths were predicted using the DISL approach. The maximum depth of damage was determined to be 1.92 m, using the -1 standard deviation properties. For the modelled cases, the average depth of damage was determined to be 1.0 m. The explicit incorporation of anisotropic stiffness in the DGR models could lead to decreased damage depth prediction. However, the conservative approach is desirable for nuclear waste purposes.

9.9 Acknowledgements

The authors would like to thank the Nuclear Waste Management Organization of Canada (NWMO) and the National Science and Engineering Research Council of Canada for supporting this research financially. The discussions with the NWMO staff and their comments regarding this topic are also greatly appreciated. Ontario Power Generation kindly provided the Niagara Tunnel Project data to the authors for their previous research, and the continued use of the data is greatly appreciated.

9.10 References


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Chapter 10: General Discussion

10.1 Applications to Repository Design in Sedimentary Rock

A nuclear waste repository is a system of components, natural and man-made, acting together to isolate the repository horizon from the surrounding underground environment and the biosphere. The major components of a repository are illustrated in Figure 10.1 and include; access and ventilation shafts, horizontal adit developments, and emplacement room or boreholes. The access and ventilation shafts pass from surface, through the isolating rock mass, and end at the repository horizon. The horizontal adit developments are long tunnels for access and from which boreholes are drilled to provide the waste canister receptacles. The canister itself is an isolating component, designed to withstand high compressive stresses and to minimize the rate of corrosion of the canister. The canister will typically be surrounded by low permeability bentonite or a similar material to act as the first barrier after the canister. The interface between the bentonite and the rock mass, as well as the excavation damage zone represent a potential flow pathway for radionuclide migration when the canister eventually deteriorates.

The purpose of a cut-off seal is to impede the flow of radio nuclides along the periphery of the excavation and as such must be constructed to bisect the HDZ and key into the EDZ to have the greatest effect. A variety of cut-off construction methods are available and include; saw cuts, precision line drilling, or road header, for example (Perras and Diederichs 2010). Cut-off seals will be constructed at various key locations (Figure 10.1), which will be site dependent. Conceptually each adit will have multiple cut-off seals near the intersection with the main access adit. Similarly, near the intersection of the shafts with the access adits represents a key location where multiple excavations converge to one primary exit pathway (the shaft damage zone). Shaft cut-off seals are likely to be constructed in key geological formations to minimize flow across
formational contacts. Figure 10.2 illustrates how the cut-off keys into the rock mass to intersect the damage zone and impede flow.

The research presented in this thesis can be applied to the design of underground repositories in sedimentary rocks. The numerical indicators used to define the dimensions of the EDZs can also be used to evaluate cut-off design considerations, such as shape, proximity to lithological contacts, and spacing. These concepts are briefly discussed in this chapter to highlight the use of numerical modeling for design of cut-offs and to discuss some of the limitations of the continuum based approach.

Figure 10.1: Illustration of a nuclear waste repository showing the two main areas for cut-off seal construction; along the repository horizon adits near the emplacement rooms and along the access shafts to isolate the repository horizon (modified from RAWRA 2013).
Figure 10.2: Illustration of cut-offs keyed into the undisturbed rock mass (modified from Martino et al. 2007).

10.1.1 Cut-off Shape
A cut-off is intended to impede the flow of radio nuclides along the periphery of the excavation and as such must bisect the HDZ and key into the EDZ to have the greatest effect. Few researchers have focused on the stability of the cut-off excavation for shafts. The focus has predominately been related to hydrogeology, seal materials and in-situ adit experiments. To facilitate discussion of cut-off design in sedimentary rocks, preliminary cut-off shapes (Figure 10.3) were modeled using axisymmetric and three-dimensional models.
The construction of the seals has been investigated by a number of researchers. Seals can be constructed using multiple saw cuts (Olsson et al., 2004), grinders (road header as suggested by Hatch (2008)), or precision line drilling and hydraulic splitting (Martino et al., 2007). The different methods of constructing the seal slots can achieve different shapes more efficiently.

Consideration for the stress regime, rock mass properties, rock mass anisotropy, structure and construction induced damage must be taken into account on a site by site basis for excavation method selection. The typical shapes are illustrated in Figure 10.3 and variations or combinations of these are also possible.

The extent of the cut-off can be determined using the HDZ and EDZ definitions discussed in Chapter 8; however the excavation of the cut-off itself will induce fracture propagation and plastic yielding. Only a preliminary analysis has been conducted for this discussion and further study is necessary to fully determine appropriate modeling approaches.
Figure 10.4: Wedge, wedge-box, and slot cut-off geometries showing maximum stress contours and plastic yielding. For the slot cut-off plastic yield isolation between the slot tip and the shaft induced yielding is possible.
These modeling approaches will also have to investigate long term rock mass behaviour, which is discussed under future research directions. The preliminary results show that at a depth of 600 m a plastic yield zone approximately 0.5 m deep developed around a shaft in a limestone. It required a 4 m deep slot cut to totally isolate the plastic yielding at the tip of the slot from the plastic yielding around the shaft, as shown in Figure 10.4.

A wedge cut-off shape was modeled with a 2 m deep apex and shallow plastic yielding, 0.25 m deep, occurred around the entire edge of the wedge (Figure 10.4); however stress concentrations at the wedge tip would provide confinement and reduce fracture apertures and limit propagation.

Construction of a wedge cut-off which completely isolates the plastic yield zone around the excavation may only be practical under very specific conditions. Generally speaking with stresses of 15-20 MPa a wedge shaped cut-off will provide good confinement at the wedge apex to reduce fracture apertures.

The box shaped cut-off, with a sloped (2:1) roof, was modeled 1 m deep back from a shaft wall. Similar results to the wedge shaped cut-off were achieved, with a shallow plastic yield zone extending from the exterior of the cut-off (Figure 10.4). However, the box shaped cut-off has stress concentrations in the corners of the cut-off, which are less stable than the wedge apex and may cause degradation by under cutting the sides of the cut-off.

Since the slot cut-off shape showed it was possible to isolate plastic yielding at the slot tip from the plastic yielding around the shaft excavation, further analysis was conducted to demonstrate the application of the numerical indicators used to determine the EDZs and to see how these zones change due to slot construction.
Figure 10.5: Cut-off slot thickness analysis using the volumetric strain (contraction to extension transition) to indicate when the EDZi (plastic yield and volumetric extension) crosses the cut. Note that the HDS is indicated by low confinement and is tensile (positive minimum principal stress).
An analysis was conducted to progressively increase the thickness of the slot cut-off which was constructed to key into the EDZo created by the shaft excavation. At 0.25 m thickness (Figure 10.5) the volumetric strain reversal line (black line in Figure 10.5) is shown to cross the slot tip outside the plastic yield zone (bottom-left model in Figure 10.5). This means that the EDZi is cut-off effectively with the thin slot. At a slot thickness of 0.75 m, the cut-off is breached by the EDZi as indicated by the volumetric strain reversal line crossing the slot tip within the plastic yield zone (bottom-right model in Figure 10.5). This indicates that a wide box cut-off may not satisfactorily intersect the EDZi to cut-off flow in the numerical analysis.

This numerical analysis does not consider the long-term rock mass behaviour aspects of the construction cycle for cut-offs (100 years after shaft excavation) and does not consider the anisotropic nature of sedimentary rocks. Both should reduce the expansion of the damage zone due to the cut-off slot construction. The long-term rock mass behaviour should soften the rock mass such that when the cut-off is constructed the new crack propagation energy is absorbed by the already damaged rock mass. Similarly, for a vertical shaft in horizontally bedded sedimentary rocks the stress concentration at the slot tip will clamp the bedding due to increased normal stress and thereby reduce inward convergence by absorbing the strain energy on the bedding.

10.1.2 Lithological Contacts
Alternating layers of shale and limestone were simulated with axisymmetric methods and compared to cross sectional models of both shale and limestone with stresses equivalent to the contact zone at 560 m depth. In the axisymmetric model result, of Figure 10.6, a variation in the maximum shear strain contour as the excavation approaches the lithological contact is observed. In the shale there is a reduction in the depth to the 0.002 % shear strain contour and in the limestone there is an increase to the 0.0005 % contour from the background depths away from the...
lithological contact. This area of disturbance from the background depth to the shear strain contour, which closely bounds the plastic yield limit, is roughly 2.4 m either side of the lithological contact. Outside of this zone of influence, the plain strain cross sectional models could be used to infer the behaviour away from the contact zone. The plain-strain models conservatively indicate deeper plastic yielding than the axisymmetric models, as indicated by maximum shear strain contours in Figure 10.6.

Figure 10.6: Plain-strain cross sectional model for circular shaft in shale (top-left) and limestone (bottom-left) and axisymmetric, excavation advance parallel, model showing influence of lithological contact between shale and limestone on the depth of maximum shear strain contour.
For closely layered rock masses, with alternating layer stiffnesses, this approach is insufficient to capture the behaviour and three-dimensional modeling will be necessary if the normal to the layering is parallel to the long axis of the excavation. If the strike of the layering parallels the long axis of excavation then the behaviour of the multi-layered system can be captured by modeling in two-dimensions, as discussed in Chapter 9. The anisotropy of the geological conditions should be accounted for, as suggested by Perras (2009), unless lamination thickness to excavation radius ratio is greater than 0.9. Also, the stress levels can be sufficiently high and the orientation suitable such that laminations are clamped, due to elevated normal stress on the bedding, which may result in a more isotropic behaviour. Cut-offs placement away from lithological contacts should be considered to avoid disturbances due to stiffness variations.

10.1.3 Cut-off Spacing

Multiple cut-off structures are typically planned for construction along repository adits and access shafts (Hatch 2008). Where cut-offs are to be constructed next to each other, for redundancy or other purposes, the spacing should be considered in terms of the influence on the damage zones.

The construction of a cut-off slot will locally increase the EDZi. This expansion, from the background shaft EDZi depth, has been used to examine the influence of multiple cut-off slots. How the depth of the deviation from the background EDZi is determined is illustrated in the top portion of Figure 10.7. Slot spacings of 0.5 to 10.0 m were investigated with short-term cut-off construction simulation. The depth of deviation from background EDZi versus the spacing is graphed in Figure 10.5 and shows that at first the deviation increases when the two cut-offs interact creating a larger EDZi in the area of the slot tip. This interference also creates a breach across the cut-off. As the spacing increases, interference becomes less and the expansion of the EDZi across the cut-off ceases. The optimum cut-off spacing in Figure 10.7 has been shown to be
when the depth falls below the minimum disturbance at the smallest spacing. More detailed analysis and advanced rock mass constitutive models may reduce the optimum spacing.

Figure 10.7: Top shows schematic representation of the volumetric strain criteria (depth above background at the midpoint) used to evaluate the spacing between consecutive cut-off slots. Bottom graph shows spacing verses depth above background with optimum window starting at a depth below the minimum spacing.
10.2 Limitations of the Continuum Approach

The approach of this thesis has been to examine continuum methods and the application for predicting the dimensions of the EDZs. Certain limitations do exist with this approach and the application of discontinuum methods is discussed to address some of these limitations.

10.2.1 Discrete Structures

A comparison between isotropic brittle methods and anisotropic methods using joint elements to simulate bedding has been completed in this thesis (Chapter 9). The joint elements are a means to include discrete structure in the otherwise continuum model. Computing capacity on standard desk tops can allow for a variety of joint networks to be incorporated into a continuum model at scales of 10’s of centimeters. A study by Day et al. (2013) indicates that by increasing the complexity of a continuum model with the addition of joint networks, can change the deformation geometry from the purely continuum model. By incorporating a variety of joint orientations, randomly using a voronoi network, at a sufficiently small scale, failure can be controlled by the joint elements instead of the continuum component in between the elements. This can be used to determine susceptible joint orientation for failure, but complete detachment or new fracture generation (in the continuum) is not possible. In this case discontinuum methods or hybrid methods are required.

10.2.2 The Highly Damaged Zone

The HDZ is a zone of interconnected, open fractures around the excavation. Incorporating an appropriate dilation parameter into a continuum code increases the volumetric expansion and therefore displacement over a model with zero dilation. A numerical study by Walton and Diederichs (in press) was able to reproduce in-situ deformations measurements between 10-20 mm, which should typically be sufficient in stiff rock masses.
Even though it is possible to capture adequate convergence with continuum methods, the geometry and orientation of fractures within the HDZ is of interest so that fluid transport along discrete fractures can be analyzed. Typical repository depths may also exceed the rock mass spalling limit and therefore crack propagation and removal of confinement using discontinuum models will reflect more accurately the failed geometry. The generation of fractures absorbs energy in the system and therefore discontinuum and hybrid codes should yield smaller dimensions of the EDZs over the conservative continuum predictions. Discontinuum or hybrid codes should be evaluated and compared to continuum models to determine if suitable benefits are gained by increasing the complexity of the numerical models being employed.

10.3 Recommended Research Directions
The research presented in this thesis has focused on the understanding the nature of excavation induced damage in sedimentary rocks and on predicting the dimensions of the EDZs using continuum methods. Based on this research a number of avenues for further theoretical and applied research are suggested.

10.3.1 Fracture Generation Codes
Promising fracture generation codes, which can accurately capture rock fracture processes, are currently in a state of development for commercial applications. For sedimentary rocks the influence of bedding planes and other rock mass features (veins, etc.) on fracture propagation directions need to be incorporated into fracture generation codes. With preferred fracture propagation orientations available within numerical models, realistic fracture patterns will be generated. Combining fluid flow capability within the same fracture generation code could allow for the life cycle of a nuclear waste repository to be simulated; from excavation of the repository, cut-off construction, backfill and sealing, to re-saturation and fluid migration. This is currently
done at different scale and with different numerical methods. Once the codes are developed they will have to be calibrated to in-situ fluid flow measurements along fractures within the EDZs in order to verify that realistic numerical results are possible at the repository scale.

10.3.2 Laboratory Testing
Standards exist for both compressive (ASTM 2010) and tensile (ASTM 2008) strength determination in the laboratory, however, there is no specification that the samples used for both tests should come from the same locality and be of a similar nature (i.e. classified as the same rock type, fabric oriented correctly for the failure mechanism of the test). It should be standard practice to conduct compressive and tensile strength tests on core samples which are adjacent to each other. In this manner better correlations between intact rock properties can be made. In particular it would be useful to strengthen the proposed relationships between tensile strength and CI and direct and indirect tensile strength for a variety of specific rock types.

10.3.3 Cut-off Life Cycle Analysis
Current capabilities of numerical methods can allow for certain aspects to be suitably modeled together, such as thermo-hydro-mechanical coupling, within the repository system. For cut-off designs it will be important to study the time–dependent behaviour of the rock mass as the cut-off structure will be excavated at some point after the repository has been constructed and is operational. Research into the interaction of the backfill with the cut-off structure, as well as, fluid and gas migration within the repository should be conducted as numerical tools continue to advance.

10.4 References


Chapter 11: Summary and Conclusions

11.1 Summary of Primary Findings
In this thesis the development of excavation damage in sedimentary rocks has been investigated with a particular focus on mudrocks and carbonates. The objectives of this thesis were achieved through laboratory testing, field observations, and numerical simulations.

11.1.1 Understanding Mechanical Properties of Mudrocks and Carbonates
Part I of this thesis, including Chapters 2 through 4, investigated the variability of sedimentary rock properties, crack initiation and propagation at the laboratory and field scales, and influencing factors on crack propagation.

The sedimentary rock properties of Ontario were found to be favorably consistent within certain formation where significant data was available, such as the Queenston and Lindsay / Cobourg. The variability of the intact rock properties could be reduced by classifying the samples according to geological classifications systems introduced in Chapter 2. The most consistent property was the CI threshold, which for the Queenston Formation had a range of only 8 MPa (min to max) despite having variable amounts of silt content (when < 75%). This finding demonstrates that CI is a material property independent of influences from the testing procedure.

The true tensile strength of rock is determined by a direct tension test where the sample is pulled apart. Difficulties with preparing samples and completing valid tests mean that direct tension tests are not routinely completed. Indirect methods are often used, however, these tend to overestimate the tensile strength. It was determined that the Brazilian Tensile Strength should be reduced such that the value is comparable to the equivalent Direct Tensile Strength. The reduction was found to be rock type dependent and the factor \( f \) in \( DTS = f \cdot BTS \) was found to be
0.9, 0.8, 0.7 for metamorphic, igneous, and sedimentary rocks respectively. This understanding at
the laboratory scale was applied to the understanding of excavation damage development in
sedimentary rocks and also applied to the numerical prediction aspects of this thesis,

11.1.2 Excavation Damage Zone Characteristics in Mudrocks and Carbonates
Part II of this thesis, including Chapters 5, 6, and 7, document observations by the author and
others on the excavation damage development in mudrocks and carbonates. At both the
laboratory and field scales it was observed that bedding and calcite veins influence the crack
propagation directions; causing deviations in the expected path. At the excavation scale this
influences the failure geometry and fracture network character. The failure geometry is controlled
by the strength, stiffness and stress of the rock mass. Four stages of damage were suggested for
sedimentary rocks; crack initiation, shearing and deflection, haunch failure, and beam deflection.
The bedding creates a plane of anisotropy which influences the crack orientation. Based on
failure observations within the Queenston Formation and the Quintner limestone, as well as data
reported in the literature, the cracks within the EDZ should be primarily orientated parallel or
perpendicular to the bedding planes. There are two exceptions to this generalization; when the
cracks are close to the excavation boundary and secondly for cracks in front of the excavation
face. At the excavation boundary they tend to follow the boundary orientation. As the cracks
move away from the boundary they rotate parallel or perpendicular to bedding. In front of the
excavation face the cracks parallel the stress trajectories making a bullet or chevron shaped
fracture pattern. When the stress to strength ratio is high, unsupported spans will reach a fragile
state of equilibrium. Continued failure can occur with changes in the environment, such as
humidity, water flow, stress changes, or human influences. When the stress to strength ratio is
low, time dependent strength degradation can occur.
Understanding the development and character of the EDZ in sedimentary rocks helps to improve the predicted dimensions of the EDZs. The EDZ is an important consideration for nuclear waste repository design, as it marks a potential flow path way.

11.1.3 Predicting the Dimensions of the EDZs

Part III of this thesis, including Chapters 8 and 9, explore the use of continuum methods to predict the dimension of the EDZs, as well as the sensitivity of the models to various factors, such as tensile strength and bedding orientation.

Numerical indicators were evaluated to determine those most suitable to identify the EDZs. Plastic yielding was found to be most suitable for determining the depth of the EDZ. The volumetric strain reversal (contraction to extension) was found to be suitable to identify the depth of the EDZ. Low confinement and/or high maximum shear strain values were found to be suitable to determine the depth of the HDZ. The numerical modeling demonstrated that the empirical depth of brittle spalling (Martin 1997, Diederichs 2007) over predicts the damage depth at high stress to strength ratios.

Typical host rocks for nuclear waste repositories can typically be considered brittle rocks. Using the DISL approach (Diederichs 2007) to predict the dimensions of the EDZs is sensitive to the tensile strength used as an input to determine the peak failure envelope. Differences of up to 20% were found depending on the tensile strength used.

When the higher indirect tensile strengths are used in the DISL approach (Diederichs 2007) larger damage dimensions are predicted than when lower tensile estimates are used. Although this may seem conservative, over predicting the dimensions of the EDZ could prohibit construction due to an inability to design an adequate cut-off or could cause more damage to the
rock mass and a possible short circuit at the cut off tip due to unexpected rock conditions (i.e. EDZ smaller than expected).

The orientation of the excavation relative to the bedding and plane of anisotropy also influences the dimensions of the EDZs and needs to be considered in numerical model predictions. Back analysis of the failure at the Niagara Tunnel Project within the Queenston Formation demonstrated the importance of the anisotropic stiffness on correctly capturing both the depth of failure and chord close measurements. Forward prediction for a vertical shaft within the Queenston Formation demonstrated that when the excavation cross section is parallel to the plane of anisotropy, isotropic models give similar results to anisotropic models.

Generally, prediction using continuum methods can be used to determine the dimensions of the EDZs. It is most suitable for determining the dimensions of the EDZo and EDZi and can be used to estimate the HDZ. For the HDZ, the in-situ failure mechanisms are not captured in the continuum models. The in-situ failure mechanisms would dampen the energy released in the failure process and therefore the continuum models overestimate the dimensions of the HDZ.

Specific conclusions from each chapter are listed in the following section.

11.2 Detailed Conclusions

The following is a listing of the detailed conclusions of the key areas of this thesis.

11.2.1 Sedimentary Rock Properties

- To determine if samples for laboratory testing have been damaged during the drilling process, P- and S-wave velocities from down borehole geophysical logging can be compared to velocities measured in the laboratory before testing.

- Average point load index values can be used to estimate the UCS for preliminary design purposes and for the sedimentary rocks of Ontario. The correlation was determined to be $\text{UCS} = 24 \ i_{50}$ with an $R^2$ of 0.8.

- The CI threshold was found to be more consistent for siliciclastic rocks than for carbonates, which is likely due to the more heterogeneous nature of carbonates.
Geological classification systems are a useful way to distinguish rocks which behave differently mechanically and using these systems allows for reasonable predictions of the properties to be made.

A geomechanical classification system was developed to aid in the prediction of the mechanical behaviour of sedimentary rocks.

The Queenston and Lindsay / Cobourg Formations have uniform properties when they have been classified into the different rock types and reasonable preliminary values can be predicted for different localities based on previous testing.

11.2.2 Importance of Tensile Strength Determination

The DISL method is sensitive to the tensile strength used as an input and can result in a difference of 20% in the predicted EDZ dimensions.

Different indirect tensile tests are used in practice and typically yield higher strengths than equivalent direct tensile tests.

Reduction factors to reduce the Brazilian tensile strength (BTS) to direct tensile strength (DTS) values were determined based on main rock types, as follows:

- \( \text{DTS} = 0.9 \times \text{BTS} \) for metamorphic rocks,
- \( \text{DTS} = 0.8 \times \text{BTS} \) for igneous rocks, and
- \( \text{DTS} = 0.7 \times \text{BTS} \) for sedimentary rocks.

Preliminary estimates of tensile strength can be made using \( T = \frac{C1}{8} \cdot \frac{UCS}{T} \) gives a good approximation of \( m \), when the data is separated by specific rock types and locations.

11.2.3 Observations of Excavation Behaviour in the Queenston Mudstone

Four zones of excavation behaviour where observed at the Niagara Tunnel Project; including local instabilities (wedges), stress shadows at the Whirlpool-Queenston contact, St. Davids Gorge influences, and high regional horizontal stresses.

The stress regime is largely controlled by the depth of the Niagara River Gorge and St. Davids Gorge.

Overbreak reached consistently 4 m in depth in the high stress areas within the Queenston and formed a stress induced notch.

The damage development begins with micro cracking parallel to bedding and deflection cannot processed unless the haunch fails into the excavation. This is followed by bed detachment due to brittle fracture parallel to bedding and deflection above beds in the tunnel crown.
- Fractures generally occurred parallel to bedding above the crown or below the invert and perpendicular to bedding elsewhere.
- Chevron or bullet shaped fracture patterns due to stress orientation at the excavation face were observed, similar to other excavations in claystones and clayshales.

11.2.4 Observations of Excavation Behaviour in the Quintner Lime Mudstone

- A variety of failure mechanisms in limestone exist and depend on spacing of bedding, strength, karstic features, stress magnitude, and stress orientation.
- At Gonzen mine, as the depth of overburden increased, the failure mechanism changed from gravity driven to stress driven failure where stress were concentrated.
- Observed failure at Gonzen mine included; pillar instability due to overstress, roof collapse due to relaxation, spalling at corners and intersecting adits, and stope collapse.
- Schmidt hammer rebound energy can be used to determine the geometry of the yield zone, which could be useful in determining the extent of the damage zone at the excavation surface and is a method to verify the stress orientation based on the location of the damage.
- Calcite veining was found to influence crack propagation during UCS testing and influenced the geometry of spalled areas in the field.
- CI was the factor that was least influenced by calcite veins in the UCS samples during testing.
- Numerical back analyses based on field observations suggest that some aspects of the failure modes can be captured numerically.
- Some failure modes, such as the stope collapse, are not adequately explained with empirical or numerical models and a time-dependent aspect is likely influencing the failure process.

11.2.5 Conceptual EDZ Character in Sedimentary Rock

- Bedding and anisotropy influence the orientation of cracks within the EDZs.
- Cracks generally form parallel or perpendicular to bedding.
- When the stress field is oriented such that the normal stress on bedding planes is increased, the EDZ dimensions will be smaller than when the stress is parallel to bedding.
- The EDZ geometry is influenced by the stress ratio and magnitude. When the stress ratio is close to 1, the geometry will be more square and when the ratio is greater than 1, a stress notch will form (influenced by the bedding orientation as above.
11.2.6 Predicting EDZ Dimensions using Continuum Models

- In continuum models, an EDZo and EDZi can be identified when post-peak properties are applied.
- EDZo is determined as the maximum extent of plastic yielding.
- EDZi is determined as the volumetric strain reversal point (contraction to extension) within the plastic yield zone.
- HDZ is determined as the low confinement zone with high maximum shear strain.
- Based on a variety of stress scenarios and rock properties, best fit equations were determined to describe the damage zone dimensions for circular excavations as follows:
  - \( \text{EDZo} / a = 1 + 0.6(\pm0.07) (\sigma_{\text{max}}/\text{CI} – 1)^{0.6(\pm0.04)} \)
  - \( \text{EDZi} / a = 1 + 0.4(\pm0.07) (\sigma_{\text{max}}/\text{CI} – 1)^{0.5(\pm0.07)} \)
  - \( \text{HDZ} / a = 1 + 0.2(\pm0.06) (\sigma_{\text{max}}/\text{CI} – 1)^{0.7(\pm0.25)} \)
- Statistical methods are useful to examine the range of possible EDZ dimensions.

11.2.7 Anisotropy and Excavation Orientation

- For the back analysis of the Niagara Tunnel, incorporating anisotropic stiffness and strength (Laminated Anisotropy Method – LAM) was important to capture both the depth of failure and the chord closure measurements.
- A normal to shear stiffness ratio of 1-2 used with the joint elements was found to result in numerical chord closure close to those measured.
- The DISL model can capture the depth of failure and using dilation can produce chord closure measurements within the same magnitude of those measured.
- The dilation parameter, \( m_d \), of 2 resulted in the lowest chord closure measurements, which was still above that measured at the tunnel.
- When the excavation cross section is parallel to the plane of anisotropy, isotropic modeling is sufficient to capture the dimensions of the damage zones and anisotropic models yield similar results.

11.3 Contributions

The scientific contributions developed as part of this thesis are represented in Chapters 2 through 9. Chapter 4 has already been accepted for publication and two others have already been submitted for publication (Chapters 5 and 8) and others will be modified and submitted (Chapters
All contributions made as part of this research have been summarized below.

11.3.1 Articles Published in a Refereed Journal


11.3.2 Articles Submitted to Refereed Journals


11.3.3 Articles in Preparation


11.3.4 Fully Refereed Conference Papers and Presentations (Perras as first author only)


11.3.5 Reports Produced from Research Activities


11.3.5.1 Posters Produced from Research Activities


11.3.6 Invited Presentations


11.3.7 Courses Taught

[34] GEOL 413 - Rock Engineering for 4th year undergraduate students. Includes lectures, and examinations prepared by Matthew Perras.
[35] GEOL 300 – Field School for 2nd year undergraduate students, including field school manual and appendix to the manual, which were revised by Matthew Perras from the previous year to accommodate changes in the course.

11.3.7.1 Course Manuals

Appendix A: Sedimentary Rock Properties of Southern and Eastern Ontario
Appendix A.1: Details of the data from the literature summarized in Table 3.2

Table A.1.1: Values reported in the literature, by source and rock class, for the Unconfined Compressive Strength (UCS) of sedimentary rocks from Southern and Eastern Ontario, including the, average (Avg), minimum (Min), maximum (Max) and the number of samples tested (# Tested).

<table>
<thead>
<tr>
<th>Source</th>
<th>Rock Class</th>
<th>Avg UCS (MPa)</th>
<th>Min UCS (MPa)</th>
<th>Max UCS (MPa)</th>
<th># Tested</th>
</tr>
</thead>
<tbody>
<tr>
<td>Barlow 1995</td>
<td>Crystalline Dolomite</td>
<td>114.7</td>
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<td></td>
<td>Sandstone</td>
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<td>53.0</td>
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<td>9</td>
</tr>
<tr>
<td></td>
<td>Shale</td>
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<td>1.1</td>
<td>9</td>
</tr>
<tr>
<td>Craig 2003</td>
<td>Crystalline Dolomite</td>
<td>104.7</td>
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<td>158</td>
</tr>
<tr>
<td>Dusseault and Loftsson 1985</td>
<td>Shale</td>
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<td>6</td>
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<tr>
<td>Ghazvinian et al. 2013</td>
<td>Lime Mudstone</td>
<td>93.3</td>
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<td>6</td>
</tr>
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<td>Golder and Associates 2011</td>
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<td>Holm 1986</td>
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<td></td>
<td>Packstone</td>
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<td>Shale</td>
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<td>Lo and Cooke 1989</td>
<td>Shale</td>
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<td>Lo and Hori 1979</td>
<td>Crystalline Dolomite</td>
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<td>Lo et al. 1987</td>
<td>Shale</td>
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<td>Rogers 1982</td>
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<th>Source</th>
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<td>Wai and Lo 1982</td>
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### Table A.1.2: Values reported in the literature, by source and rock class, for the Crack Initiation (CI) of sedimentary rocks from Southern and Eastern Ontario, including the, average (Avg), minimum (Min), maximum (Max) and the number tested (# Tested).

<table>
<thead>
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<th>Source</th>
<th>Rock Class</th>
<th>Avg CI (MPa)</th>
<th>Min CI (MPa)</th>
<th>Max CI (MPa)</th>
<th># Tested</th>
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### Table A.1.3: Values reported in the literature, by source and rock class, for the Crack Propagation (CD) of sedimentary rocks from Southern and Eastern Ontario, including the, average (Avg), minimum (Min), maximum (Max) and the number tested (# Tested).

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<th>Source</th>
<th>Rock Class</th>
<th>Avg CD (MPa)</th>
<th>Min CD (MPa)</th>
<th>Max CD (MPa)</th>
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### Table A.1.4: Values reported in the literature, by source and rock class, for the Brazilian Tensile Strength (BTS) of sedimentary rocks from Southern and Eastern Ontario, including the, average (Avg), minimum (Min), maximum (Max) and the number tested (# Tested).

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<thead>
<tr>
<th>Source</th>
<th>Rock Class</th>
<th>Avg BTS (MPa)</th>
<th>Min BTS (MPa)</th>
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<td>Craig 2003</td>
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Table A.1.5: Values reported in the literature, by source and rock class, for the Young’s Modulus \( (E_i) \) of sedimentary rocks from Southern and Eastern Ontario, including the, average (Avg), minimum (Min), maximum (Max) and the number tested (# Tested).

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<th>Source</th>
<th>Rock Class</th>
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<td>Lo et al. 1987</td>
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Appendix A.2: Details of the data from the DGR testing reports shown in Table 3.3

Table A.2.1: Values from DGR testing reports, by rock class, for the Unconfined Compressive Strength (UCS) of sedimentary rocks from Southern and Eastern Ontario, including the, average (Avg), minimum (Min), maximum (Max) and the number of samples tested (# Tested).

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<th>Min UCS (MPa)</th>
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Table A.2.2: Values from DGR testing reports, by rock class, for the Crack Initiation (CI) of sedimentary rocks from Southern and Eastern Ontario, including the, average (Avg), minimum (Min), maximum (Max) and the number tested (# Tested).

<table>
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<th>Min CI (MPa)</th>
<th>Max CI (MPa)</th>
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Table A.2.3: Values from DGR testing reports, by rock class, for the Crack Propagation (CD) of sedimentary rocks from Southern and Eastern Ontario, including the, average (Avg), minimum (Min), maximum (Max) and the number tested (# Tested).

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Table A.2.4: Values from DGR testing reports, by rock class, for the Brazilian Tensile Strength (BTS) of sedimentary rocks from Southern and Eastern Ontario, including the, average (Avg), minimum (Min), maximum (Max) and the number tested (# Tested).

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Table A.2.5: Values from DGR testing reports, by rock class, for the Young’s Modulus ($E_i$) of sedimentary rocks from Southern and Eastern Ontario, including the, average (Avg), minimum (Min), maximum (Max) and the number tested (# Tested).

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Appendix A.3: Details of the data from this thesis shown in Table 3.4

Table A.3.1: Values from the current study, by rock class, for the Unconfined Compressive Strength (UCS) of sedimentary rocks from Southern and Eastern Ontario, including the, average (Avg), minimum (Min), maximum (Max) and the number of samples tested (# Tested).

<table>
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<th>Min UCS (MPa)</th>
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Table A.3.2: Values from the current study, by rock class, for the Crack Initiation (CI) of sedimentary rocks from Southern and Eastern Ontario, including the, average (Avg), minimum (Min), maximum (Max) and the number tested (# Tested).

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<th>Min CI (MPa)</th>
<th>Max CI (MPa)</th>
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Table A.3.3: Values from the current study, by rock class, for the Crack Propagation (CD) of sedimentary rocks from Southern and Eastern Ontario, including the, average (Avg), minimum (Min), maximum (Max) and the number tested (# Tested).

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Table A.3.4: Values from the current study, by rock class, for the Brazilian Tensile Strength (BTS) of sedimentary rocks from Southern and Eastern Ontario, including the, average (Avg), minimum (Min), maximum (Max) and the number tested (# Tested).

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<th>Max BTS (MPa)</th>
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Table A.3.5: Values from the current study, by rock class, for the Direct Tensile Strength (DTS) of sedimentary rocks from Southern and Eastern Ontario, including the, average (Avg), minimum (Min), maximum (Max) and the number tested (# Tested).

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<th>Max DTS (MPa)</th>
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Table A.3.6: Values from the current study, by rock class, for the Young’s Modulus (E<sub>i</sub>) of sedimentary rocks from Southern and Eastern Ontario, including the, average (Avg), minimum (Min), maximum (Max) and the number tested (# Tested).

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Appendix A.4: Details of the data used in Figures 3.15 and 3.16

Table A.4.1: Unconfined Compressive Strength (UCS) values showing the average (Avg), minimum (Min), maximum (Max), and number of samples tested (# Tested) for each formation and rock class, as shown in Figure 3.15.

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Table A.4.2: Young’s Modulus ($E_i$) values showing the average (Avg), minimum (Min), maximum (Max), and number of samples tested (# Tested) for each formation and rock class, as shown in Figure 3.15.

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<th>Min $E_i$ (MPa)</th>
<th>Max $E_i$ (MPa)</th>
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Table A.4.4: Brazilian Tensile Strength (BTS) values showing the average (Avg), minimum (Min), maximum (Max), and number of samples tested (# Tested) for each formation and rock class, as shown in Figure 3.16.

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