A CONTINUUM MODELLING STUDY OF MACRO-GEOMETRY CHOICES AND THEIR IMPACT ON EXCAVATION DAMAGE ZONE DEVELOPMENT IN BRITTLE ROCK AT DEPTH

by

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Abstract

Canadian and International experts agree that deep underground repositories are the single best solution regarding safe, long-term nuclear waste disposal (IAEA 2009). Numerous European and Asian countries have already begun extensive research and preliminary design analyses associated with site assessment for high-level fuel storage facilities known collectively as deep geological repositories (DGRs) (ANDRA 2005, Armand et al. 2003, NAGRA 2002, Martino and Chandler 2004). In Canada, Kincardine, ON, is currently the proposed site that will host the low- to medium-level nuclear waste generated from the Bruce, Pickering and Darlington nuclear power plants. The project conceptualization, as well as the specific sites, have undergone extensive public critique and analyses. The project is in the final licensing stage of the process.

To ensure long term stability and safety of the DGRs, the prediction and understanding of the excavation damage zone (EDZ) around the associated shafts, placement room tunnels, storage voids and access tunnels is paramount. The general EDZ consists of component zones of damaged, fractured and influenced rock moving radially away from the center of the excavation. The outer “influenced” zone consists of rock that has been elastically strained, while the “damaged” zone undergoes small scale, discontinuous crack damage, subsequently increasing the permeability of the surrounding rockmass. The “fractured” zone is located nearest to the excavation wall/face, where continuous (connected) fractures dominate.

Numerous details within a preliminary design of the tunnels and intersections within a DGR will influence the development and ultimate impact of the EDZ on ultra-long-term repository safety. For the various DGR tunnelling projects that include the placement room tunnels, access tunnels and vertical shafts, details of shape will dictate the ultimate support demands and safety requirements. Using case examples, as well as 2-dimensional and 3-dimensional continuum numerical modelling simulations, this thesis will illustrate key impacts of
macro-geometry choices, such as shape and dimensions of placement room and access tunnels, as well as details, which include corner geometry, aspect ratio, excavation sequence and methodology, on EDZ and highly damaged zone (HDZ) development in underground infrastructure.
Co-Authorship

This thesis is the cumulative product of the research conducted by Sarah E. Cain. While some of the ideas, as well as scientific and editorial comments and figures, were guided by Dr. Mark Diederichs, with additional contributions from members of the Queen’s Geomechanics Group, the written content is solely that of the author.
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<th>Description</th>
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<tr>
<td>AECL</td>
<td>Atomic Energy Canada Limited</td>
</tr>
<tr>
<td>APM</td>
<td>Adaptive Phase Management</td>
</tr>
<tr>
<td>CD</td>
<td>Crack Damage Threshold <em>or</em> Critical Damage Limit</td>
</tr>
<tr>
<td>CDZ</td>
<td>Construction Damage Zone</td>
</tr>
<tr>
<td>CI</td>
<td>Crack Initiation Threshold</td>
</tr>
<tr>
<td>CWFS</td>
<td>Cohesion Weakening Friction Strengthening</td>
</tr>
<tr>
<td>DGR</td>
<td>Deep Geological Repository</td>
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<tr>
<td>DISL</td>
<td>Damage Initiation Spalling Limit</td>
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<tr>
<td>EDZ</td>
<td>Excavation Damage Zone</td>
</tr>
<tr>
<td>EDZ_o</td>
<td>Outer Excavation Damage Zone</td>
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<tr>
<td>EDZ_i</td>
<td>Inner Excavation Damage Zone</td>
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<tr>
<td>EIZ</td>
<td>Excavation Influence Zone</td>
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<tr>
<td>FDEM</td>
<td>Finite-Discrete Element Method</td>
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<tr>
<td>FDM</td>
<td>Finite Difference Method</td>
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<tr>
<td>FEA</td>
<td>Finite Element Analysis</td>
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<td>FEM</td>
<td>Finite Element Method</td>
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<tr>
<td>GSI</td>
<td>Geological Strength Index</td>
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<tr>
<td>H-B</td>
<td>Hoek-Brown Failure Criterion</td>
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<tr>
<td>HCB</td>
<td>Highly Compacted Bentonite</td>
</tr>
<tr>
<td>HDZ</td>
<td>Highly Damaged Zone</td>
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<tr>
<td>ISRM</td>
<td>International Society of Rock Mechanics</td>
</tr>
<tr>
<td>K_{H,h}</td>
<td>Horizontal Stress Ratio</td>
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<tr>
<td>mBgs</td>
<td>Meters Below Ground Surface</td>
</tr>
<tr>
<td>M-C</td>
<td>Mohr-Coulomb Failure Criterion</td>
</tr>
<tr>
<td>Acronym</td>
<td>Full Form</td>
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<tr>
<td>NWMO</td>
<td>Nuclear Waste Management Organization</td>
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<tr>
<td>PDE</td>
<td>Partial Differential Equation</td>
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<tr>
<td>RMR</td>
<td>Rockmass Rating</td>
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<tr>
<td>SKB</td>
<td>Swedish Nuclear Fuel and Waste Management Company</td>
</tr>
<tr>
<td>UCS</td>
<td>Unconfined Compressive Strength</td>
</tr>
<tr>
<td>UFC</td>
<td>Used Fuel Canister</td>
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<tr>
<td>URL</td>
<td>Underground Research Laboratory</td>
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Chapter 1: Background Information

1.1 Deep Geologic Repositories Within the Canadian Content

Both Canadian and International experts agree that an underground repository is the single best solution for ultra-long-term storage of nuclear waste, both from a longevity and safety perspective (IAEA 2009). Numerous countries, including France, Switzerland, Sweden, Finland and China have carried out extensive research, including the construction of underground laboratories in candidate host rocks, and preliminary design analyses associated with site assessment for high-level, in-rock, spent fuel storage facilities known collectively as deep geological repositories (DGRs) (ANDRA 2005, Armand et al. 2003, NAGRA 2002, Martino and Chandler 2004).

1.1.1 Canadian Deep Geologic Repository

In Canada, Kincardine, ON, is currently the proposed site that will host the low- to medium-level nuclear waste generated from the Bruce, Pickering and Darlington nuclear power plants. The project is in the final licensing stage of the process. This repository safety case is founded on a stable near field and far field geosphere as noted in Al et al. (2011). The near field considerations include excavation damage zone (EDZ) management around repository rooms as well as the vertical shafts (in this case through various layers of sedimentary rock).

For the safe disposal of Canada’s high-level nuclear waste, an “Adaptive Phase Management” (APM) plan has also been undertaken by the Nuclear Waste Management Organization (NWMO) and approved by the Canadian Government in 2007. The APM consists of three major components; the high-level nuclear waste repository itself, which is anticipated to be located at roughly 500 m – 700 m depth, the associated surface facilities, and the used fuel transport system (Boyle and Meguid 2015, Crowe et al. 2015).
1.1.1.1 Canadian DGR Geometry, Layout and Configuration

Two vertical shafts will provide access to the DGR, which consists of an extensive horizontal layout encompassing centrally located access drifts, as well as perimeter drifts. Perpendicularly oriented tunnels will serve as the entry points to the used fuel canister (UFC) placement room tunnels. These placement rooms will form a laterally extensive and vast array of panels (see Figure 1-1 and Figure 1-3).

![Diagram showing conceptual layout and Mark II placement configuration](image)

**Figure 1-1:** Conceptual layout and Mark II placement configuration (Boyle and Meguid 2015, Crowe et al. 2015).

The initial DGR design, labelled as the “Mark I”, was largely based on the design employed by the Swedish Nuclear Fuel and Waste Management Company (SKB) (Carvalho and Steed 2012). An arched or half-elliptical profile was used for the main room design, along with a linear and spaced array of vertical boreholes drilled into the floor. The vertical holes were
designed to house the UFCs. Both the holes and the placement rooms above would be packed with bentonite. This configuration can be seen in Figure 1-2.

More recently, an alternative design, known as the “Mark II”, has been developed for preliminary design of high-level repositories. This version (shown in Figure 1-1 and again in Figure 1-2) no longer employs the in-floor borehole design, while also substituting the half-elliptical placement/storage room tunnels for a rectangular profile (Crowe et al. 2015, Guo 2016). The fuel canisters would be placed across the width of this placement room in a staggered fashion surrounded by compacted bentonite. These placement rooms, measuring 3.2 m in width and 2.2 m in height, will measure 304 m in length and employ a lateral spacing of roughly 25 m. In total, the Mark II DGR design will include eight placement rooms panels, each consisting of placement rooms and support pillars; drill-and-blast excavation will be used (Crowe et al. 2015, Radakovic-Guzina at al. 2015). Figure 1-2 shows the design iterations of the Canadian DGR design, where option #1 corresponds to “Mark I”, option #2 corresponds to “Mark IIb” and option #3 corresponds to the current “Mark II” design.

Figure 1-2: Design options for Canada’s future high-level nuclear waste DGR (termed APM) (after Crowe et al. 2015).
The cross-sectional geometries of the two placement room designs (Mark I and Mark II) are compared below in Figure 1-3.

**Figure 1-3: Left:** Dimensions of the Mark I placement/storage rooms (modified from Carvalho and Steed 2012), with the transport tunnel above and the placement borehole beneath. **Right:** Dimensions of the Mark II placement/storage rooms (modified from Guo 2016).

With a footprint of roughly 4 km\(^2\) (1.9 km by 2.0 km), the DGR will contain 4.6 million used CANDU fuel bundles upon completion, placed and safely stored within the UFCs shown in Figure 1-1. These fuel bundles are made up of fuel pellets, within which the natural uranium UO\(_2\) is housed. The NWMO describes these UFCs as a “mid-sized capacity copper/steel composite vessel containing 48 used fuel bundles” each (NWMO 2015, Radakovic-Guzina et al 2015). Figure 1-4 on the following page displays a conceptual footprint of the Mark II DGR design.
Figure 1-4: Plan-view conceptual layout of the proposed Mark II APM DGR design. Note that this design is for construction within sedimentary rock (NWMO 2015).

The UFCs will be stored within rectangular buffer boxes, filled with highly compacted bentonite (HCB). These buffer boxes will be stacked in an offset, staggered fashion, with dense backfill blocks occupying the space between the buffer boxes (NWMO 2015, Radakovic-Guzina et al. 2015). Figure 1-5 highlights the buffer box and dense backfill placement configuration within the placement room tunnels.
Figure 1-5: Placement room configuration showing the stacked nature of the UFC buffer boxes (blue) and dense backfill blocks (grey) (NWMO 2015). Left side corresponds to the dead-end of the placement room tunnel.

An EDZ cut-off seal will also be constructed near the opening of the placement room; the premise and objective behind the EDZ cut-off seal has been described in Chapter 1.3.3. Upon placement of the buffer boxes and backfilling of the placement room tunnels, the entrance to each placement room will be sealed-off with HCB and a concrete bulkhead (NWMO 2015, Radakovic-Guzina et al. 2015). This configuration can be seen below in Figure 1-6.

Figure 1-6: Plan view (top) and longitudinal view (bottom) of the Mark II DGR placement rooms, as well as the locations of the EDZ cut-off seal (labelled as “room seal”) and concrete bulkhead (NWMO 2015).
To reduce the amount of exposure pathways, as well as to reduce the amount of required backfilling and seal construction, each placement room is a dead-end tunnel. As previously stated, a main shaft and secondary service vertical shafts will provide access to the DGR, along with a third shaft used for ventilation (all shown in Figure 1-4). These shafts will require backfilling and multiple seals (termed cut-offs) for both the shaft excavations and any significant EDZ generated around the shafts. A detailed understanding of the mechanics and nature of this EDZ is required for minimization of EDZ and management during closure.

1.1.2 Canadian DGR Site Selection Process

An in depth and iterative site selection process has been employed since the APM plan’s conception in June of 2007. With a primary focus on safety, the planning process began in late 2008. The NWMO highlighted that the sites must meet or exceed all regulatory requirements, while a large focus of the process was also placed on engaging with communities to gather a sense of their interest in hosting the DGR. The NWMO provided information to build awareness with these communities regarding the project, while also facilitating question periods with community members. Upon receiving interest in the DGR from a potential host community, initial screening took place, with more in-depth preliminary assessments carried out for the communities that passed the screening process and were deemed potentially suitable. An array of both sedimentary and crystalline geological host sites was considered during this planning phase (Crowe et al. 2015).

In May of 2010, the site selection process began. In total, 22 communities expressed interest in hosting the DGR facility; three were in northern Saskatchewan, and the remaining 19 in Ontario. Of these 22 communities, 21 passed the initial screening process titled “Step 2” in 2012; the selection criteria are described thoroughly in “Moving Forward Together: Process for
Upon completion of initial screening processes carried out in 2012, as well as feasibility studies conducted in 2013, 17 potential host communities were selected for further assessment. This screening process was based on numerous factors inducing geological suitability, repository design, transportation costs, community well-being and socio-economics (Crowe et al. 2015).

Figure 1-7: December 2014 status of the communities that had expressed interest in hosting the DGR (Crowe et al. 2015).

Following desktop studies that aimed to quantify the host sites’ geological suitability and the potential for the project to contribute to the well-being of the host community, the 17 sites were narrowed down to 13 by the end of 2014 (Crowe et al. 2015). Figure 1-7 above highlights the communities still involved in the selection process as of December 31 of 2014.
The NWMO is currently undertaking a detailed technical site evaluation of the nine communities that successfully passed the initial screening process (Step 3). This technical site evaluation is based on six fundamental factors. Firstly, the site must provide safe underground containment and isolation of the nuclear waste. The site must be resilient to any future geological processes and/or dramatic climate changes that might affect the long-term life span of the DGR. Thirdly, the nuclear waste must be completely isolated from any current and future human activity. Furthermore, the site must present favorable conditions to carry out a detailed site characterization and enable on-going data collection, while also facilitating safe construction, operation, closure and on-going monitoring of the DGR. Lastly, the site must be accessible by safe and secure roads (Crowe et al. 2015).

As of 2017, the NWMO has and continues to carry out airborne geophysical surveys, detailed geological mapping and borehole drilling. It is anticipated that the detailed selection process will not highlight a preferred single site until 2023 at the earliest (Crowe et al. 2015).

1.1.3 Canadian DGR Host Rock – Crystalline versus Sedimentary

The nine sites currently under review to serve as the host for the DGR present two different geological host conditions. As illustrated in Figure 1-7, the three sites in south-western Ontario are location within sedimentary rock, while the six sites in northern Ontario lie within the crystalline Canadian Shield.

1.1.3.1 Sedimentary Host Rock Properties

The Paleozoic sedimentary rocks of the Michigan Basin eastern flank play host to the three south-western Ontario sites. Located at a depth of 500 – 700 mbgs (meters below ground surface) (depending on site) within this sedimentary sequence is the Cobourg Limestone unit, the proposed host rock for the three sedimentary sites. The actual depth of the repository will depend on geologic characteristics at the selected site, along with other design features and safety
considerations. Figure 1-8 below displays a geological cross section of the Michigan Basin; note that the borehole “DGR-2” corresponds to the approximate location of the three, closely located, potential host communities in south-western Ontario (shown previously in Figure 1-7).

Figure 1-8: Vertically exaggerated geological section of the Michigan Basin. This subsurface geology represents the host conditions at the three south-western Ontario potential host communities/sites (Frizzell et al. 2008). The 3DGF (3D geological framework) boundary and DGR2 borehole indicated refer to the geological studies related to the intermediate level DGR near Kincardine (Al et al 2011).

This horizontally layered argillaceous limestone is approximately 27 m thick and is greater than 600 km in extent (as shown in Figure 1-8). Natural barriers/layers lie above and below the Cobourg limestone; it is capped by a 200 m think shale layer (Frizzell et al. 2008, NWMO 2015). This geology is presented in Figure 1-9 on the following page.

Of significant importance with respect to safe storage of nuclear waste is its low potential for lateral contaminant transport. The Cobourg Limestone and its enclosing rock units present low
permeabilities (on the order of $10^{-4}$ m/sec) and hydraulic conductivities, minimal cross-unit flow and show little evidence of glacial influence (NWMO 2015).

The area surrounding the three south-western Ontario sites is seismically quiet, geomechanically stable and presents a groundwater table that is isolated to surface. The depth at which the DGR will be constructed is expected to be virtually dry (NWMO 2015). These attributes are fundamental components of the future DGR site.

Figure 1-9: Subsurface stratigraphy at the thee south-western Ontario potential host sites (NWMO 2015).
Members of the Geomechanics Research Lab at Queen’s University have carried out various laboratory tests on the Cobourg Limestone unit in order to further investigate its potential as a host rock. A list of the mechanical properties of the Cobourg limestone can be found in Chapter 2.4. These properties will serve as modelling inputs for the finite element (FEM) and finite difference (FDM) modelling results that will be discussed in further chapters of this thesis.

1.1.3.1.1 Insitu Stress Regime

The future high-level APM nuclear waste DGR is expected to be located a depth of between 500 – 700 mbgs, depending on the selected site conditions.

Numerous insitu stress estimates have been compiled to generate likely stress values at the DGR depth. This database includes data presented by Adams and Bell (1991) pertaining to insitu stresses of the St. Lawrence Platform, stress gradient surveys collected by the Canadian In-Situ Stress Database (complied and presented in Arjang 2001) with reference to the proposed APM DGR sites, and insitu stresses at the Norton Mine collected via the United States Bureau of Mines overcoring (Bauer et al. 2005). At a depth of 670 mbgs within the Columbus Limestone Unit, this data suggests the stress regime at the future DGR site is as follows (Radakovic-Guzina et al 2015). Note that although the Columbus Limestone unit, belonging to the Early to Middle Devonian period, presents differing material properties, its depth and location within the Michigan Basin is a good analog for predicted stresses within the Cobourg Limestone.

\[
\sigma_{v,3} = 22.5 \text{ MPa} \\
\sigma_{H,1} = 36.7 \text{ MPa (N75°W)} \\
\sigma_{R,2} = 28.3 \text{ MPa}
\]

These values support the understanding that the horizontal stresses in the St. Lawrence Platform exceed the vertical stresses at any of the proposed DGR sites. Given the horizontal stress values presented above, horizontal stress ratios (\(K_h\) and \(K_H\)) values of 1.25 – 1.63 are expected at
the DGR site; that is, the horizontal stresses are 1.25 – 1.63 times greater than the vertical stress at the DGR depth.

These stress levels must be considered in terms of rockmass instability and plastic damage, which will result in some crack initiation and fracture propagation around the excavation. These stress levels can be graphically correlated to the expected rockmass damage in relation to the information and data presented in the following chapter (Chapter 2.5).

1.1.3.2 Crystalline Host Rock Properties

The Canadian Shield and its crystalline rock presents an alternate geological site for the proposed DGR. It is anticipated that a repository at one of the six potential northern Ontario sites would be housed at 500 m depth. Similar to the three south-western Ontario potential sites, this portion of the Canadian shield is seismically quiet and stable. Note that the material and strength properties of the crystalline host rock lie outside of the scope of this thesis, and will not be part of the analyses discussed in the following chapters.

1.2 Categorizing Excavation Induced Damage

To properly classify the degree and extent of damage induced around the various underground excavations, it is vital to understand the terminology associated with excavation induced damage.

The basic premise of the EDZ categorization lies in the fact that the density and connectivity of fractures decrease as we move radially outwards from the surface of the excavation (Perras and Diederichs 2016). Before discussing the EDZ in detail, however, it is important to note that the construction damage zone (CDZ), immediately surrounding the excavation wall, is independent of the EDZ. Although sometimes impossible, the construction-induced damage around an underground excavation can be mitigated by altering the excavation method to better suit the insitu conditions (Siren et al 2015).
The generalized EDZ, which can develop independent of excavation methodology, is composed of concentric annuli (Siren et al. 2015; Lanyon 2011; Tsang et al. 2005; Emsley et al. 1997) which are classed as the innermost “highly damaged zone” (HDZ), the inner and outer EDZ, and the outermost excavation influence zone (EIZ). These are represented schematically in Figure 1-10 below.

**Figure 1-10:** Damage zones corresponding to the DISL brittle modelling approach by Diederichs (2003, 2007). This figure is also based on the cohesion weakening friction strengthening (CWFS) approach according to Martin (1997), Kaiser et al. (2000), Hajiabdolmajid et al. (2002), Diederichs (2003) and Diederichs et al. (2004).

The outermost region defines the mechanical limit of influence of the excavation. The excavation influence zone or EIZ consists of rock that has been elastically strained (Perras and Diederichs 2016). While this zone has been referred to as EdZ (excavation-disturbance zone) by Tsang et al. (2006) and others, the impact is limited to elastic damage and not to disturbance as
per the more conventionally accepted definition. It should also be noted that for low porosity rocks, the impact on permeability in this zone is insignificant.

The outer “damaged” (EDZ\textsubscript{o}) zone undergoes small scale crack damage with isolated, unconnected fractures, subsequently increasing the permeability of the surrounding rockmass by up to 10x. The inner “fractured” zone (EDZ\textsubscript{i}) contains partially-continuous fractures that increase the bulk permeability by 10x to 100x. It should be noted that measurable dilation (inelastic volumetric strain) is only observed within EDZ\textsubscript{i} (Perras and Diederichs 2016).

Immediately adjacent to the excavation at higher stress levels, there may exist a zone referred to as the highly damaged zone (HDZ). Due to the stress-induced damage within the HDZ, coupled with structural and geometric effects, the rockmass in this zone typically presents a high degree of interconnected fractures, while spalling is also commonly observed (Perras and Diederichs 2016). These fractures pose a serious issue since they result in irreversible damage and further promote fluid conductivity through the “fractured” zone (Massart and Selvadurai 2014; Bossart et al. 2002; Souley et al. 2001), with equivalent permeability increases greater than four orders of magnitude.

In terms of the mechanics of stress induced damage (in the various EDZs in Figure 1-10), it is helpful to consider the stress space plotted on the right of Figure 1-10. If the excavation does not move the stress state into tension, or above the compressive “crack initiation threshold” defined as CI (Diederichs and Martin 2010), no damage can occur; only an EIZ is present. If tensile rupture is generated, or shear failure takes place at higher confinements, an HDZ will form. Due to the effects of extension crack propagation at lower confining stresses, HDZ in the form of spalling can occur near the excavation boundary at stresses below the yield envelope for lab samples (critical damage limit or CD). This is shown by the non-linear “S-shaped” curve bounding the red zone in Figure 1-10. The physics behind this S-shaped curve, damage initiation,
propagation and the spalling limit is discussed extensively by Diederichs (2003, 2007) and Diederichs et al. (2004).

Between damage initiation at CI and yield at CD, a zone of progressive distributed damage exists (EDZ₀ to EDZᵢ) with fracture intensity and connectivity increasing at higher differential stress.

While the schematic in Figure 1-10 shows a circular excavation, this thesis will focus on geometric effects on the extent and magnitude of the HDZ, as well as the inner (“fractured”) and outer (“damaged”) portions of the EDZ.

1.3 The Impacts of Excavation Damage Zone Development on Nuclear Waste Placement Rooms, Storage Caverns and Shafts

1.3.1 Importance of Understanding and Predicting EDZ Development

An in-depth understanding of EDZ development about nuclear waste shafts, placement room tunnels and storage caverns is paramount with respect to the safe design, construction and maintenance of a DGR. It is important to understand the implications of this EDZ development to modify and optimize DGR component designs prior to construction. An understanding of the anticipated EDZ magnitude in relation to excavation methodology can help ensure that excavation techniques are not inducing larger-than-predicted fracture propagation and rockmass damage. The installation of sufficient support for safe operation during the construction and used-fuel placement stages of the DGR lifespan is also dependent on a thorough understanding of the anticipated EDZ development. Finally, modelling of long-term EDZ effects will also aid in developing a better understanding of the predicted long-term behaviour of the rockmass over the safe storage life of the used-fuel.
1.3.2 Implications of an EDZ on the Stability of Underground Excavations

1.3.2.1 Increased Fracture Propagation and Connectivity

It is understood that EDZ development within these underground tunnels can lead to increased fracture propagation and connectivity.

Recall that the basic premise of the EDZ categorization lies in the fact that the density and connectivity of fractures decrease as we move radially outwards from the surface of the excavation (Perras and Diederichs 2016); the EDZ is sub-divided into two sections known as the EDZ_o and EDZ_i. The EDZ_o undergoes small scale crack damage with isolated, unconnected fractures that subsequently increases the permeability of the surrounding rockmass by up to 10x, while the EDZ_i contains partially-continuous fractures that increase the bulk permeability by 10x to 100x. As previously discussed, the outermost region that defines the mechanical limit of influence of the excavation is the EIZ, which consists of rock that has only been elastically strained.

If the excavation results in an increase in brittle rockmass failure and further promotion of exposure pathways for crack propagation and connectivity, the radial extent of the EDZ will further increase. Moreover, the extent of permeable rockmass is directly correlated to the extent of the EDZ. An increase in the EDZ thereby results in a radial increase in irreversibly-damaged rockmass, which dramatically impacts the premise of the multi-barrier DGR system that relies on intact rockmass, in addition to the engineered barriers, to safely contain the nuclear waste and retard migration, inhibit microbial activity around the UFC and prevent/mitigate long-term rockmass strength degradation, as well as gas and heat migration and expansion. It is also worth noting that increased support measures would need to be undertaken to properly support the damaged rockmass to ensure safe working conditions during the construction and used-fuel placement stages of DGR project.
Minimizing fracture propagation and connectivity is thereby vital in reducing the extent of plastic brittle rockmass failure that dictates the extent of the EDZ.

1.3.2.2 Brittle Rockmass Spalling

Spalling can be defined as “the development of visible extension fractures under compressive loading near the boundary of an excavation” (Diederichs et al. 2010). It is a form of non-violent brittle rockmass failure by which slabs of rock flake or “spall” from the solid rockmass forming the excavation boundary. Spalling primarily occurs in rockmasses with an index of greater than 12, a Geological Strength Index (GSI) of greater than 65, and a ratio of unconfined compressive strength (UCS) to tensile strength (T) of greater than 15 (Diederichs 2007). Note that at lower UCS/T ratios, shear failure mechanisms dominate; this is of less concern given the EDZ is induced around deep underground excavations within low confinement stress regimes. Each of these aforementioned brittle spalling failure criteria hold true for the Cobourg Limestone unit (see Chapter 2.4 for description of material and strength properties). The following graph (Figure 1-11) concludes that although the Cobourg Limestone (with a UCS of 113 MPa and a T of 5 MPa) is prone to spalling (medium to high spall potential ratio), it is only susceptible to non-violent brittle rockmass failure with low rockburst (strain burst) potential (Diederichs 2007).

It is also worth noting that spalling is typically associated with deep excavations within relatively massive and unfractured rockmasses (Martin 1997), thereby providing further correlation to the Cobourg Limestone unit. This is highlighted by the plotting of the spalling potential of the Cobourg Limestone in Figure 1-11 on the following page.
As previously described, extension crack propagation at lower confining stresses can cause spalling near the excavation boundary at stresses below the yield envelope for lab samples (previously described as the critical damage limit or CD shown in Figure 1-10 of Chapter 1.2). This is supported by the understanding that the wall rock tends to fail at a stress of 40 - 50% of the UCS obtained from lab testing (therefore approximately 45 – 57 MPa for the Cobourg Limestone). Additionally, the laws of fracture mechanics dictate that cracks within the rockmass cannot grow past a $\sigma_3/\sigma_1$ ratio of greater than 0.1; this defines the spalling limit.

Based on the insitu stress conditions of the proposed possible DGR sites, wherein the horizontal stresses exceed the vertical stresses ($\sigma_h > \sigma_v$), minor brittle rockmass spalling is anticipated to occur in the walls of the excavation. Furthermore, it is predicted that stress convergence about the tunnel crown and floor will result in excess spalling leading to overbreak in the form of v-shaped notches.
1.3.2.3 Increased Spalling Leading to Tunnel Overbreak

As previously understood, brittle rockmass failure around an underground excavation can be described as spalling. As per Martin 1997, “the resulting failed zone is commonly referred to as breakout, dog-ear, or V-shaped notch”. Another term commonly used to describe the zone of failed rockmass is “overbreak”.

Although overbreak may be excavation induced (i.e. the resulting of blasting), it is also known to be the result of stress convergence/concentration about an excavation boundary, causing increased boundary stresses orthogonal to the major primary stress (i.e. 90° to $\sigma_1$). (Martin 1997). High stress regimes can lead to large overbreak and tunnel collapse. This overbreak results in the formation of v-shaped notches.

Numerous studies investigating the response of brittle rock to underground openings have been conducted. As the ratio of tunnel stress to UCS increases, so does the brittle rockmass spalling and resulting overbreak (Diederichs 2007). With the use of continuum modelling of a circular test tunnel, Martin et al. (1997) demonstrated how brittle rock failure generates v-shaped notches aligned with the orientation of the maximum compressive stress. Tensile regions were also observed at 90° to the v-shaped notches (predicted minor spalling regions in the tunnel walls of the DGR). As noted by Hajiabdolmajid et al. (2002), the slabbing process that generates these v-shaped notches terminates upon achieving a more stable geometry. The authors attributed this phenomenon to the resulting increase in confinement and a subsequent decrease in plastic strain (which correlates to induced damage), thereby stopping the cohesive loss within the rockmass, ultimately leading to the termination of the v-shaped notch formation.

Given the fact that the stress regime at the possible future DGR sites presents greater horizontal stresses as compared to the vertical stress, this overbreak is expected to therefore occur in the crown and floor of the excavations.
Figure 1-12: Brittle rockmass spalling leading to overbreak in the crown and roof of a test tunnel at Canada’s former Underground Research Laboratory (URL) in Pinawa, Manitoba (Martin 1997).

It is also worth considering the practical implications of overbreak within the DGR excavation. This increased volume of damaged rockmass will result in an increase in project time and cost due to the need for additional support measures. The increased volume of failed rockmass will require that the overbreak void be filled with shotcrete or cast concrete prior to the installation of the final tunnel lining.

1.3.3 Management of EDZ Development in Underground Excavations

1.3.3.1 Tunnel Shape and Profile Details

It is widely understood that circular profiles act to promote favourable stress redistribution about underground excavation as compared to tunnel geometries with right angle corners, such as squares or rectangles. In relation to Canadian DGR research, testing conducted at the Pinawa, Manitoba URL concluded that an ovaloid tunnel profile, with its long axis oriented parallel to the maximum principal stress, presented the most favourable stress distribution about
the opening (Read 2004). This reduction in stress magnitude can be directly correlated to a decrease in the plastic radius about an excavation, thereby playing a significant role in the extent of the EDZ.

Results presented within later chapters of this thesis will also aim to support this conclusion by assessing the impacts of various tunnel geometries and corner details on the resulting EDZ development.

1.3.3.2 EDZ Cut-offs/Seals

To properly manage the development of EDZ about the DGR shafts, tunnels and placement room tunnels, various EDZ cut-off designs can be used to mitigate the magnitude of brittle rockmass failure. The cut-off research program was implemented by the NWMO in 2014 to explore the geometry and configuration of EDZ cut-offs and their contribution to minimizing potential EDZ development and exposure pathways (Perras et al. 2015). The study aimed to develop a criterion for selecting cut-off geometry based on rockmass properties and insitu stress regimes for both vertical (shaft) and horizontal (tunnels and placement caverns) DGR openings.

The premise of these EDZ cut-offs lies in the excavation of slots or keys in the DGR component (shaft, tunnel or cavern) at a perpendicular orientation to the excavation axis. This excavation aims to limit the propagation of extension fractures along the tunnel axes, thereby reducing the extent of the potential EDZ pathway. The cut-offs are then sealed with either bentonite clay and/or concrete.

1.3.3.2.1 EDZ Cut-offs/Seals in Vertical Shafts

Perras et al. (2015) explored the impacts of rectangular EDZ cut-off slots on the resulting decrease in EDZ about vertical DGR shafts. As shown in the schematic on the following page (Figure 1-13), the thickness and spacing of these cut-off slots were assessed in terms of the resulting radius of plastic damage, as well as the fracture connectivity and density within zones
between each cut-off slot; recall that the EDZ$_i$ and EDZ$_o$ define the boundaries between partially-continuous fractures and isolated, unconnected fractures. The transition from contraction to extension (by measuring volumetric strain) was used to quantify the effectiveness of the cut-off slots.

![Figure 1-13: Schematic highlighting the relation of cut-off slot thickness and spacing to the resulting HDZ and EDZ development about a DGR shaft (Perras et al. 2015).](image)

The results of this study concluded that a rectangular cut-off slot is the best suited geometry to effectively isolate the plastic yield about the DGR shaft, while also minimizing the connection of damaged rockmass across each cut-off slot. It was also concluded that a cut-off slot thickness to depth/extent of plastic radius ratio of 0.13 will best isolate the regions of extensile volumetric strain, while employing multiple cut-off slots spaced at either less than 0.5 m or between 4.0 – 6.0 m present the most favourable results regarding EDZ reduction for a shaft measuring 5 m in diameter (Perras et al. 2015).
1.3.3.2.2 EDZ Cut-offs/Seals in Horizontal Access Tunnels and Placement Rooms

Additional research investigating the impacts of cut-off/seals in horizontal access tunnels and placement rooms of DGRs has also been carried out. Similar to the aforementioned design of the rectangular cut-offs in vertical shafts, these cut-offs again aim to limit the propagation of extension fractures along the tunnel/cavern axes to ultimately reduce the potential EDZ pathways.

The design of the cut-off seals employed in the horizontal tunnels of the DGR are quite different from the rectangular profile used in the vertical shaft. The NWMO has carried out preliminary research that concludes that a triangular/wedge, notch-shape cut-off presents the most favorable results in terms of the reduction in fracture propagation and EDZ development along the length of the horizontal tunnel. Preliminary findings suggest that the triangular/wedge-shaped ring successfully acts to interrupt the continuity of fractures along the length of the placement tunnels, thereby minimizing the extend of the EDZ. By reducing the radial extent of fractured and damaged rockmass, the safety factor of both the placement tunnels and DGR as a whole are positively impacted. Figure 1-14 is an example of the triangular/wedge, notch-shape EDZ seal.

Figure 1-14: Continuum modelling (FLAC³D) set-up of the triangular/wedge-shaped EDZ seal (green) along the DGR placement tunnel (red) (Radakovic-Guzina et al. 2015).
As highlighted in Figure 1-6, the EDZ seal will be located between the access tunnel (right) and storage location of the used fuel canisters (to the left) within the placement tunnels/rooms. Upon placement of used fuel canisters and backfilling of the placement tunnels, the EDZ seal will be backfilled with bentonite clay (similar to the void space between the canisters in the placement rooms).

1.4 Summary of Thesis Variables (Part I)

This chapter aims to summarize the thesis variables that will be examined based on the introductory information presented in Chapter 1 through 1.3. Each of the variables listed below will be examined on an interconnected level, with the aim of assessing both their individual and combined impact on EDZ development (magnitude, size, profile etc.) about underground nuclear waste excavations of varying shapes and dimensions.

1.4.1 Placement Room and Cavern Shape Details

The profile and shape details of the used fuel storage tunnels are significant to the overall safety factor of the DGR. Based on a well-developed understanding of stress redistribution about circular versus rectangular excavations, 2D tunnel profile details must be considered to achieve the most stable DGR excavations and ensure long-term safety.

In these projects, a great deal of attention is paid to the various levels of damage induced around the excavation during construction and over the long-term operating (>100 years) and safe storage life (up to a million years). Other tunnelling applications, such as fuel storage and hydraulic tunnelling, also share this critical consideration of brittle rock damage. The 3D development of damage around an advancing tunnel also has implications for support staging.

Based on the previous understanding that circular profiles act to promote favourable stress redistribution about underground excavation as compared to tunnel geometries with right angle corners, a 2D and 3D assessment of this conclusion has therefore been conducted.
Results presented in this thesis will aim to support the conclusion drawn at the Pinawa, Manitoba URL, which proved that an ovaloid tunnel profile presented the most favourable stress distribution about the opening (Read 2004). As will be discussed in later chapters, this confirmation will be carried out by assessing the impacts of various tunnel geometries and corner details on the resulting EDZ development.

Rectangular, horseshoe and circular tunnel profiles have all been comparatively assessed, along with a study that assesses the degree of rounding (about the tunnel corners) and its correlation to anticipated EDZ reduction about underground excavations. Based on the understanding of favourable stress redistribution resulting from circular tunnel profiles, it is predicted that 2D rounding of the rectangular tunnel corners will reduce the EDZ development and propagation about the tunnel perimeter at both the face and along its entire length. The exact dimensions of the tunnel geometries will be presented in each of the related chapters.

1.4.2 Insitu Stress Regime

1.4.2.1 Summary of Predicted DGR Insitu Stresses

As previously discussed in Chapter 1, Canada’s future high-level DGR (at one of the three south-western Ontario potential host sites) is expected to be located a depth of between 500 and 700 mbgs, depending on the selected site conditions. Chapter 1 also summarizes previous research pertaining to the expected insitu stress conditions at a depth of roughly 670 mbgs. Literature suggests that the three anticipated/likely principal stresses are as follows:

\[ \sigma_{v,3} = 22.5 \text{ MPa} \]

\[ \sigma_{H,1} = 36.7 \text{ MPa} \ (N75^\circ W) \]

\[ \sigma_{h,2} = 28.3 \text{ MPa} \]

Additionally, horizontal stress ratios (\(K_{h,H}\) values) of 1.25 – 1.63 are expected at the three south-western Ontario potential site.
1.4.2.2 Insitu Stress Values Used for Modelling Purposes

An understanding of the predicted/likely insitu stress values at the three south-western Ontario potential host sites was used to devise a set of reasonable and conservative insitu stress values and horizontal stress ratios.

Given the density ($\rho$) of the Cobourg Limestone unit, which is roughly 2,700 kg/m$^3$, as well as gravitational loading ($g$), the vertical stress within the DGR will lie between 13.2 MPa – 18.5 MPa. Note that even though the subsurface at the proposed DGR sites presents numerous stratigraphic layers (as shown in Figure 1-8 and Figure 1-9 of Chapter 1) (NWMO 2015), this value is obtained by using an overburden of uniform/homogeneous unit weight for modelling simplicity.

The following formula was used to calculate the vertical stress, denoted by $\sigma_v$. Since the vertical stress is exceeded by both horizontal stresses, the vertical stress also doubles as $\sigma_3$.

\[
\sigma_{v,3} = \rho \cdot g \cdot z
\]

This insitu vertical stress range corresponds to estimates and values from numerous data sources summarized and explained in Chapter 1 (summarized by Radakovic-Guzina et al. 2015).

As it is understood that the horizontal stresses exceed the vertical stresses at the proposed DGR sites (Radakovic-Guzina et al. 2015), the in- and out-of-plane horizontal stresses (denoted by $\sigma_H$ and $\sigma_h$) will serve as $\sigma_1$ and $\sigma_2$ modelling inputs, respectively. The horizontal stresses are a product of the in- and out-of-plane horizontal stress ratios and the vertical stress, and are calculated as follows:

\[
\sigma_{H,1} = K_H \cdot \sigma_v
\]
\[
\sigma_{h,2} = K_h \cdot \sigma_v
\]

While published data presented in the previous Chapter 1 suggests that the horizontal stress ratios at the locations of the potential south-western Ontario DGR sites lie between 1.25 – 1.63, a range of 1.5 – 3.5 has been used for the purposes of this thesis. This increased range aims
to depict a suitable and conservative range within the overall Canadian context given the fact that the APM DGR site has yet to be selected (Brown and Hoek 1978, Al et al. 2011). Given this increased range, it is thereby expected that the horizontal stresses at the variable depth of the DGR will vary between 19.9 MPa and 64.9 MPa.

Research conducted by Itasca in conjunction with the NWMO suggests that extreme, yet rare, seismic ground motion presents minimal effect in terms of rockmass damage (NWMO 2015). As such, seismic events have not been modelled for this thesis investigation.

**1.4.2.3 Implications of Placement Room Orientation**

In addition to the impacts associated with tunnel shape and profile details on the resulting EDZ development about the tunnel perimeter and face, it is also vital to consider the tunnel orientation with respect to the major and minor in-plane horizontal stresses (given the DGR insitu stress conditions).

Testing conducted at the Pinawa, Manitoba URL also proved that tunnel orientation is also a determining factor in resulting EDZ development. As previously discussed, it was concluded that the ovaloid tunnel profile, with its long axis oriented parallel to the maximum principal stress ($\sigma_1$), presented the most favourable stress distribution about the opening/perimeter (Read 2004).

With the aim of supporting this conclusion, the various tunnel geometries used for the purposes of this thesis have therefore been assessed at two different orientations:

1. Tunnel axes aligned parallel to the direction of the major in-plane horizontal stress ($\sigma_1$)
2. Tunnel axes aligned perpendicular to the direction of the major in-plane horizontal stress ($\sigma_1$) (i.e. parallel to $\sigma_2$)

Additionally, tunnel orientation has also been assessed in terms of the resulting yield ahead of an advancing tunnel face.
Finally, these results have been combined with the data pertaining to tunnel shape and profiles details in order to provide recommendations in terms of the most favourably combined tunnel profile/shape, orientation and excavation methodology.

1.4.3 Blast Round Length and Excavation Methodology

The NWMO has stated that all placement room tunnels will be excavated using convention drill-and-blast methodology (Crowe et al. 2015, Radakovic-Guzina at al. 2015). Given the fact that an exact blast round length has not yet been specified by the NWMO, various blast round lengths have been used for the 3D modelling of the placement room tunnels for the purposes of this thesis. This study thereby aims to determine the impact of blast round length on the resulting EDZ development about the tunnel perimeter and along its entire length, as well as the resulting yield ahead of the tunnel face.

Additionally, a comparative study assessing the impact of different excavation methodologies has also been conducted. Chapter 8 of this thesis presents and discusses the significance of 3D modelling results of conventional drill-and-blast excavation versus continuous excavation (aimed to mimic the results generated by using a tunnel boring machine or TBM).

1.4.4 Delineating the Extent of the EDZ Sub-Regions

Chapter 1 discusses the importance of properly classifying the degree and extent of damage induced around the underground excavations, such as the DGR placement room tunnels. An in-depth understanding of EDZ development about underground nuclear waste excavations is vital in terms of safe design, construction and maintenance of a DGR.

It is understood that EDZ development can lead to brittle rockmass spalling, overbreak and increased fracture propagation and connectivity. This behaviour is known to cause an increase in the radial extent of the permeable rockmass, which is understood to be directly correlated to the extent of the EDZ. Given the fact that the EDZ is sub-divided into the
aforementioned EDZ₀ and EDZᵢ regions, in which the permeability is noted to increase by up to 10x and by 10x to 100x, respectively (Perras and Diederichs 2016), portions of this thesis will aim to delineate these regions, as well as their association with tunnel shape, profile details and tunnel orientation. This thesis will also aim to correlate the extent of the EDZ with respect to tunnel shape and profile details, as well as to tunnel orientation.
Chapter 2: Technical Review

2.1 Failure Criterion

2.1.1 Hoek-Brown Failure Criterion

2.1.1.1 Original Conception

The Hoek-Brown failure criterion was developed by Hoek and Brown (1980) to provide accurate input parameters for computational modelling of hard-rock, underground excavations. Hoek’s contributions include information pertaining to brittle failure of intact rock samples (via triaxial testing) (Hoek 1968), while Brown’s work consisted of studying the behaviour of jointed rock masses (Brown 1970). The failure criterion was initially developed considering intact rock properties, with the later incorporation of jointed rock mass results to better represent in-situ conditions. The failure creation is defined by the major and minor effective stresses (\(\sigma'_1\) and \(\sigma'_3\)), as well as the uniaxial compressive strength of the intact rock sample (\(\sigma_{ci}\)). Two material constants are also used (\(m\) and \(s\)), where \(s\) is equal to 1 for intact rock. The defining equation of the original Hoek-Brown failure criterion is as follows:

\[
\sigma'_1 = \sigma'_3 + \sigma_{ci} \left( m \cdot \frac{\sigma'_1}{\sigma_{ci}} + s \right)^{0.5}
\] (1)

Hoek and Brown also decided it would be beneficial to use Bieniawsk’s Rock Mass Rating (RMR) (Bieniawsk 1973, 1976, 1989) to provide a correlation of their results with in-field observations of a given rock mass. Modifications were later made to the failure criterion to better consider geotechnical issues. These modifications include the consideration of disturbed and undisturbed rock masses and re-evaluation of the tensile strength reduction for very poor quality rock masses. However, it has become apparent that the use of the Hoek-Brown failure criterion posed some issues in industry (most notably those involving slope stability) due to the lack of
inclusive normal and shear stress variables that better lend themselves to solving geotechnical problems.

As such, Bray (as summarized by Hoek 1983), along with Ucar (1986) and Londe (1988), derived the previous equation (1) to establish an exact numerical relationship between the major and minor principal stresses (and uniaxial compressive strength), and the more practically applied normal and shear stresses at failure. By using Bray’s work, which included derivations to determine tangent lines to the Mohr envelope, Hoek was then able to provide the industry with subsequent derivations for equivalent friction angle and cohesive strength parameters.

2.1.1.2 Generalized Hoek-Brown Failure Criterion

The generalized Hoek-Brown failure criterion was conceived in the mid-1990s following further research regarding tangent fitting to the Mohr envelope and determination of the cohesive strength parameter (Hoek 1994).

One of primary changes that Hoek and Brown made to the original work was the substitution of the RMR classification system with the use of the GSI (Hoek 1994). This change was made to better correlate the Hoek-Brown failure criterion to field observations. It was also proven and explained that GSI is far more capable of accurately classifying weak rockmass, as compared to the RMR system. The authors noted that the relationship between RMR and Hoek-Brown rockmass constants m and s deteriorates for weak rockmasses with an RMR < 30. (Hoek et al. 1988, Marinos and Hoek 2000, Marinos and Hoek 2001). The following sections explain the other changes made to the original Hoek-Brown failure criterion.

2.1.1.2.1 Disturbance Factor

The disturbance factor was devised to provide an inclusive mean of considering the state of the rockmass. The degree of “disturbance”, denoted by D, is based upon blast-induced damage, as well as stress relaxation following excavation. This value ranges from 0 to 1, whereby 0 is
used to describe an “undisturbed” rockmass, while highly “disturbed” rockmasses are allocated a value of 1 (Hoek et al. 2002).

2.1.1.2.2 Material Constants

The following equation highlights another change that was made to Equation 1 (Hoek 1994). Note that material constant m has been substituted with \( m_b \).

\[
\sigma_1' = \sigma_3' + \sigma_{ci}(m_b \frac{\sigma_3'}{\sigma_{ci}} + s)^{0.5}
\]  

(2)

Where \( m_b \) is defined by the following Equation 4, whereby \( m_i \) is also a material constant (Hoek 1994).

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

(3)

The generalized Hoek-Brown theory also brought about a change to the square root used in the original Equation 1 (Hoek et al. 1992, Hoek 1994, Hoek and Brown 1997). It was concluded that variable \( \alpha \), which adjusts the shape of the principal stress plot and Mohr envelope, was far more accurate. Equation 4 therefore sees the following change (Hoek 1994):

\[
\sigma_1' = \sigma_3' + \sigma_{ci}(m_b \frac{\sigma_3'}{\sigma_{ci}} + s)^\alpha
\]  

(4)

Note that variables \( s \) and \( \alpha \) are both rockmass constant defined by their own equations (Hoek and Brown 1997, Hoek 1994).

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

(5)

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

(6)

2.1.2 Mohr-Coulomb Failure Criterion

As the most widely used failure criterion in the field of geotechnical engineering, the Mohr-Coulomb failure criterion uses a linear relationship between normal and shear stress (at yield) to define the strength properties of a given material (Coulomb 1776). This relationship can
also be displayed in terms of major and minor principal stresses at failure. The following
Equation 8 is used to define the normal and shear stress relationship:

$$\tau' = c' + (\sigma_n')(\tan\phi')$$ (7)

Where $\tau'$ is the shear strength at failure for a given normal stress ($\sigma_n'$), $c'$ is the cohesive
strength and $\phi'$ is the friction angle. Note that this relationship is also known as the “direst shear
formulation”.

As shown by Equation 8, the triaxial formulation can be used to describe the relationship
between the major and minor principal stresses (Hoek 1994, Hoek et al. 2002).

$$\sigma_1' = \left(\frac{2c'\cos\phi'}{1 - \sin \phi'}\right) + \left(\sigma_3' \frac{1 + \sin \phi'}{1 - \sin \phi'}\right)$$ (8)

Where the major and minor principal stresses at failure are denoted by $\sigma_1'$ and $\sigma_3'$,
respectively.

### 2.1.2.1 Achieving Equivalent Mohr-Coulomb Failure Criterion

To properly assess the comparative results of modelling with both aforementioned failure
criteria, it is important to recall that Mohr-Coulomb utilizes a linear failure envelope, while the
generalized Hoek-Brown approach utilizes non-linear failure criterion.

The equivalent Mohr-Coulomb parameters are based on a line-of-best over a
representation range of principal stresses. The assumption is that; $\sigma_t < \sigma_3 < \sigma_{3,max}'$, where $\sigma_t$ is
the tensile strength of the intact sample (Hoek et al. 2002). The selection for the upper $\sigma_{3,max}'$
limit of confining stress is primarily based on modelling results.

The following plot (Figure 2-1) displays the graphical relationship between the
generalized Hoek-Brown and equivalent Mohr-Coulomb failure criteria. Note that Equation 8 is
also used to describe the equivalent plot.
Chapter 2 explains the detailed process and tools used to obtain the generalized Hoek-Brown failure criterion, as well as the equivalent Mohr-Coulomb failure criterion for the brittle rockmass under study. An explanation of the process for linearizing the Hoek-Brown failure criterion to create an equivalent Mohr-Coulomb failure envelope is explained by Hoek et al. (2002).
2.2 Numerical Modelling Software

2.2.1 RocScience Software RS² Version 9.0 and RS³ Version 1.0

2.2.1.1 Introduction

For the purposes of this thesis, two continuum modelling programs have been utilized. RS² (RocScience 2016: RS² version 9.0), a 2-dimensional finite element method (FEM) program with continuum modelling, and RS³ (RocScience 2016: RS³ version 1.0), a 3-dimensional continuum FEM modelling program, were used. Although capable of modelling various support elements, all the models discussed in this thesis were left unsupported to best depict the real-life scenario associated with ultra long-term underground storage of nuclear waste. While shafts, storage caverns and placement rooms within the DGR will be supported, this support will merely provide logistical safety to personnel and equipment during the construction and placement phase of the project. The limited lifespan of support elements such as reinforced shotcrete, wire mesh, forepoles, rockbolts and steel beams means that the support elements will deteriorate and cease to function or provide support long before the DGR reaches its terminal point in the project lifespan.

2.2.1.2 Finite Elements Method (FEM)

As previously mentioned, both RocScience programs use implicit FEM continuum modelling to solve any given model (RocScience 2016). FEM, also referred to as finite element analysis (FEA), is a numerical method used to approximate solutions for boundary value problems. Using known boundary conditions, coupled with differential equations applied to satisfy the boundary conditions, a solution to a boundary value problem can be determined. The formulation of a FEM model therefore results in a system of algebraic equations (Mase 1970, Bower 2011).
The models are subdivided, or discretized, into smaller parts, known as finite elements. Each element contains its own set of partial differential equations (PDE), which are approximated by using numerical methods rather than analytical solutions (which are not possible given the RocScience model geometry). The FEM thereby generates approximate equations and/or values for all the unknowns at each discrete point within the model. To solve the overall problem and model itself, these approximate equations and/or values are then combined to form a larger, global system of equations that represent the model problem in its entirety (Mase 1970, Bower 2011).

2.2.2 Itasca Software FLAC\textsuperscript{3D} Version 5.0

2.2.2.1 Introduction

Itasca’s FLAC\textsuperscript{3D} version 5.0 (Itasca Consulting Group Inc. 2012: FLAC\textsuperscript{3D} version 5.0), a 3-dimensional modelling program that uses the explicit finite different method (FDM) and performs Lagrangian analyses to solve numerical models, was also used for the purposes of this thesis (Itasca Consulting Group Inc. 2012).

2.2.2.2 Finite Difference Method (FDM)

Given a set of initial/boundary values, the FDM is a numerical method that solves a set of differential equations by approximating them with difference equations. As explained by Itasca, “every derivative in the set of governing equations is replaced directly by an algebraic expression written in terms of the field variables (e.g., stress or displacement) at discrete points in space; theses variables are undefined within elements.” (Itasca Consulting Group Inc. 2011, Itasca Consulting Group Inc. 2012). Similar to FEM, FDM is also a discretization method since finite differences are used to approximate solutions of partial differential equations.

As previously stated, FLAC\textsuperscript{3D} uses an explicit solution method, meaning that the state of a model at a future time is calculated based on the current state of the model. Conversely, implicit
methods involve solving equations that incorporate both the current and future state of the model (Itasca Consulting Group Inc. 2011, Itasca Consulting Group Inc. 2012).

The explicit nature of FLAC\textsuperscript{3D} can be represented by the following formula:

\[ f(t + \Delta t) = F(Y(t)) \]

Where \( Y(t) \) it’s the current model state of and \( Y(t + \Delta t) \) is the future state of the model. The time step is represented by \( \Delta t \) (Itasca Consulting Group Inc. 2011, Itasca Consulting Group Inc. 2012).

This time step process can be easily represented by the following Figure 2-2 and explained with the following steps:

1. Velocities and displacements are derived from stresses and forces by using the equations of motion.
2. Strain rates are derived from velocities.
3. New stresses are derived from strain rates.
4. The cycle repeats.

Each loop of the circle in Figure 2-2 corresponds to a single time step in the FDM model. The most important aspect of the system is that each box in Figure 2-2 updates each of its grid variables from known values, which remain fixed/unchanged while at the stage in the loop. As explained in the Itasca FLAC manual, “the lower box takes the set of velocities already calculated and, for each element, computes new stresses. The velocities are assumed to be frozen for the operation of the box (i.e. the newly calculated stresses do not affect the velocities)” (Itasca Consulting Group Inc. 2011, Itasca Consulting Group Inc. 2012). This assumption holds true given that fact that a very small time-step is used during each loop. “This time step is so small that information cannot physically pass from one element to another in that interval” (Itasca Consulting Group Inc. 2011, Itasca Consulting Group Inc. 2012).
The basic premise of this loop system employed by FLAC and FLAC\textsuperscript{3D} lies in the fact that the “calculation wave-speed” will always be quicker than the “physical wave-speed” and that the equations used in each of the aforementioned steps are always based on known values that remain fixed during the calculation conducted in each box of Figure 2-2. The extremely small time-step also ensures that errors in the final result are bounded (Itasca Consulting Group Inc. 2011, Itasca Consulting Group Inc. 2012).

2.3 Brittle Rockmass Failure Mechanics and Computational Applicability

The following Chapter examines the premise of brittle rockmass failure mechanics, and how the true theory differs from its application within FEM and FDM computation models.
2.3.1 Brief Introduction to Brittle Rockmass Failure Mechanics Within Underground Excavations

As explained by Martin et al. (1999), the “failure of underground openings in hard rocks is a function of the insitu stress magnitudes and the characteristics of the rock mass”, which is defined by the intact strength of the rock, as well as the surrounding fracture network. Damage around underground excavations at lower insitu stresses are controlled by the pre-existing/inherent fracture network within the rockmass. As in the case with Canada’s future DGR, higher insitu stresses result in the development of new fractures around an underground excavation; these fractures are oriented parallel to the excavation boundary and are known as “brittle failure” (Martin et al. 1999). As shown in the Figure on the following page (Figure 2-3), as the depth increases, so does the corresponding depth of fracture propagation, as well as the size of the excavation boundary that is exposed to brittle failure.

Recall that since the horizontal stress ratios within Canada range from 1.5 to 3.5, a DGR located a depth of between 500 – 700 mbgs would present a $\sigma_1$ range of 19.9 – 64.9 MPa. Given the fact that the host rock for the future DGR presents an RMR of > 75 and UCS ($\sigma_c$) of roughly 113 MPa, the insitu conditions range from intermediate to high stress magnitudes (as shown by the red box in Figure 2-3). As such, brittle rockmass failure adjacent to the excavation boundary is expected to occur within intermediate insitu stress regimes (shallower depth, lower horizontal stress ratios), while brittle rockmass failure around the entire excavation is anticipated to occur within an increased, high insitu stress regime (greater depth, higher horizontal stress ratios).
2.3.2 True Brittle Rockmass Theory versus Computational Modelling

As shown in Figure 2-3, brittle rockmass behaviour transitions from classical shear failure at high confinement to brittle spalling behaviour at low confinement (such as around an excavation).

![Table: Rockmass Failure Zones](image)

Figure 2-3: Resulting failure zone in relation to in situ stress regime and rockmass structure (Martin et al 1999, modified from Hoek et al. 1995).
In the strain hardening region of Figure 2-4, rockmass damage occurs. However, there is enough confinement that full crack propagation and connection is not achieved. For deep tunnel modelling, as in the case with the Canadian DGR, researchers and engineers are less concerned about high confinement cases (top right portion of the graphs in Figure 2-4 and Figure 2-5), and more so concerned about the lower confinement region depicted in Figure 2-4.

![Figure 2-4: Theoretical behaviour of brittle rockmasses (after Diederichs 2003).](image)

Since conventional numerical models cannot replicate the full transitional behaviour, highlighted by the solid black line in Figure 2-4, while maintaining the thermodynamics of plastic stability, two simplified envelopes are used; these are shown in Figure 2-5 on the following page. These envelopes are termed “spalling confinement limit” and “damage initiation threshold”, and will be discussed in the following section.
2.3.3 Strength Models Used to Replicate Brittle Rockmass Behaviour

While the previous section explains that brittle theory must be slightly modified in order to facilitate computational numerical modelling, it is important to note that there are two
approaches to modelling brittle failure criteria. The damage initiation spalling limit (DISL) approach, developed by Diederichs (2007), as well as the cohesion weakening friction strengthening (CWFS) implementation method, developed by Ryder and Jagger (2002) and extensively explored by Hajiabdolmajid et al. (2002), are both used and discussed throughout this thesis. The following sections explain and compare each method, as well as the numerical modelling programs to which they belong.

2.3.3.1 Damage Initiation Spalling Limit (DISL) Approach

To reproduce the non-linear “S-shaped” spalling model originally shown in Figure 2-3, Diederichs (2007) presents a simplified constitutive model to be used with analysis software that can accommodate a strain weakening constitutive model using either the generalized Hoek-Brown model (Hoek et al. 2002) or an equivalent Mohr-Coulomb approach.

In the DISL approach by Diederichs (2007), the generalized Hoek-Brown yield envelope parameters are modified as shown in Table 2-1. Used with a non-linear FEM or FDM code, the DISL model can predict brittle rock spalling, as a function of confinement, within underground excavations. The premise of the confinement dependency lies in the fact that the “damage initiation” occurs under a nominal cohesion and a low friction parameter; this is the CI envelope.

The “spalling limit” defines the region of confining stress below which crack propagation forms spalling, and above which cracks must coalesce or accumulate to form internal shearing. This limit is attained by moving towards a state of low cohesion and increased friction (Diederichs 2007).

With respect to modelling inputs, the “peak” parameters correspond to “damage initiation”, while the “residual” values depict the “spalling limit”.
Table 2-1: Equations by Diederichs (2007) to determine the appropriate DISL modelling input parameters.

<table>
<thead>
<tr>
<th>Damage Initiation (Peak)</th>
<th>Spalling Limit (Residual)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Input Parameter</td>
<td>Value/ Equation</td>
</tr>
<tr>
<td>( a_p )</td>
<td>0.25</td>
</tr>
<tr>
<td>( s_p )</td>
<td>( \left( \frac{CI}{UCS} \right)^{1/a_p} )</td>
</tr>
<tr>
<td>( m_p )</td>
<td>( s_p \left( \frac{UCS}{</td>
</tr>
</tbody>
</table>

Equivalent, Mohr-Coulomb peak and residual envelopes can also be determined to match the non-linear DISL envelopes defined by parameters in Table 2-1. The “peak” and “residual” curves for the sedimentary rock, grouped by failure criterion, have been presented in the following graph (Figure 2-6). The DISL parameters will serve as the input variables for the FEM numerical models present in this thesis.

Carter et al. (2008), explain that the residual curve is extremely sensitive to confinement. It has been confirmed that the Hoek-Brown dilation parameter “D” can be used to replicate the degree/magnitude of confinement. As in the modelling cases presented in this thesis, “for unrestrained fallout or for loose retention, dilation should be set to zero” (Diederichs et al 2004; Carter et al. 2008).

In Figure 2-6, the DISL envelopes are compared to the intact strength limit, the rockmass strength envelope for a massive rockmass (GSI of 80). The damage initiation CI thresholds are compared to 2D and 3D Griffith crack criterion as discussed in Diederichs (2003).
Figure 2-6: Mohr-Coulomb and Hoek-Brown brittle failure criterion generated with the use of the DISL modelling approach (based on Diederichs 2007).

2.3.3.1.1 DISL Approach in FEM Modelling

RocScience FEM software uses the DISL approach to model brittle rock failure. For FEM programs such as RS² version 9.0 and RS³ versions 1.0, two distinct curves/envelopes are used to model the brittle criteria. As previously explained, crack initiation is defined by the peak curve, while the spalling limit is defined by the residual curve. Note that the green lines and arrows in Figure 2-7 depict strain hardening behaviour, while the red lines and arrows represent strain softening/weakening behaviour.
Figure 2-7: Brittle criterion used in continuum FEM analyses. (A): The green line defines conventional elastic-perfectly plastic behaviour, while the red line defines elastic-brittle plastic rock behaviour. Note the two distinct envelopes: crack initiation ("peak") and spalling limit ("residual") in (B).

2.3.3.2 Cohesion Weakening Friction Strengthening (CWFS) Implementation

2.3.3.2.1 Inaccuracy of Conventional Models in Replicating Brittle Rockmass Failure

Various researchers have proven that conventional continuum models with traditional failure criteria, such as Hoek–Brown or equivalent Mohr–Coulomb, that use simultaneous mobilization of cohesive and frictional strength components cannot accurately predict or replicate the magnitude of brittle damage around underground excavation. By comparing numerical modelling results using various conventional behaviours (elastic, elastic-perfectly plastic, elastic-brittle plastic) to results from a circular test tunnel at the URL in Pinawa, Manitoba, it was determined that none of the conventional approaches were able to replicate the damage observed in the test tunnel (Hajiabdolmajid et al. 2002).
This is highlighted by the discrepancies between the “true failure zone” profile observed in the test tunnel versus the failure zone produced by the conventional models in FDM software FLAC\textsuperscript{2D} (see Figure 2-8).

2.3.3.2.2 Increased Accuracy of the CWFS Approach in Modelling Brittle Rockmass Failure

Ryder and Jagger (2002) originally adopted the constitutive model that is capable of accurately replicating brittle rockmass failure given its consideration of plastic strain-dependencies of various strength components. As explained by Hajiabdolmajid et al. (2002), “the cohesional component of strength is the predominant strength component at the early stage of brittle failure and cohesion loss is the predominant failure process leading to the observed brittle behaviour.” The authors noted that tensile cracking and eventual crack coalescence causes the degradation of cohesive strength. Another key component of the CWFS approach is that fact that

Figure 2-8: (A): Elastic-perfectly plastic constitutive model. (B): Elastic-brittle plastic constitutive model (from Hajiabdolmajid et al. 2002).
the following three criteria must be met to fully mobilize the normal stress-dependent friction strength:

1. The cohesional component of strength must be significantly reduced.
2. Excessive rockmass damage must have accumulated.
3. Rock fragments must be able to move relative to one other in shear.

The overarching aspect of the CWFS approach to implementing brittle rockmass failure mechanics is that brittle failure predominantly exhibits extensile, brittle fracturing within low-confineent regions and is characteristically defined by “the delay in frictional strength mobilization of the frictional strength” (Hajiabdolmajid et al. 2002). Simply put, brittle rockmass behaviour is defined by the formation of tensile cracks before the occurrence shear failure. The CWFS implementation approach can replicate this behaviour by using plastic strain to mobilize the tensile and shear strength behavioural components; that is, the CFWS approach is strain-dependent (Hajiabdolmajid et al. 2002). The authors described that the “delayed mobilization of the frictional strength component is attributed to the delayed development of internal normal stress (shear contact), for example, due to material heterogeneity” (Hajiabdolmajid et al. 2002).

The CWFS approach was modelled in FLAC and compared to the results observed in the test tunnel at the Pinawa, Manitoba URL. As shown in Figure 2-9 on the following page, the CFWS approach is capable of accurately replicating the true failure profile exhibited by the circular test tunnel.
Figure 2-9: CWFS model used in FLAC\textsuperscript{2D} can replicate the brittle rockmass failure exhibited in the Mine-by test tunnel (from Hajiabdolmajid et al. 2002).

Based on the strong correlation highlighted in Figure 2-9, the CWFS approach (according to Martin 1997, Kaiser et al. 2000, Hajiabdolmajid et al. 2002, Diederichs 2003, 2007 and Diederichs et al. 2004) is considered the most representative of brittle yield and damage progression.

2.3.3.2.3 CWFS Approach in FDM Modelling

The strain-hardening/softening Mohr-Coulomb constitutive model in FLAC\textsuperscript{3D} was used to implement the CWFS approach. This model differs from the classic plastic Mohr-Coulomb constitutive model in the sense that the user can allow the cohesion, friction, dilation and or tensile strength to harden or soften once plastic yield has been achieved. In the plastic Mohr-Coulomb model, each of these four parameters are fixed.

Itasca software, such as FLAC\textsuperscript{3D}, modifies internal properties as a function of strain. The strain-hardening/softening Mohr-Coulomb constitutive model code does so by measuring the total
plastic strain values (both shear and tensile strain) while the hardening parameters are modified at each iterative time-step. In this way, the pre-defined Mohr-Coulomb peak and residual properties develop over the model. As described by Itasca, “the user can define cohesion, friction and dilation as piecewise-linear functions of a hardening parameter measuring the plastic shear strain. A piecewise-linear softening law for the tensile strength can also be prescribed in terms of another hardening parameter using the plastic tensile strain.” (Itasca Consulting Group Inc. 2012)

A cohesion drop and friction mobilization occur after initial yield (CI). As such, the strain-hardening/softening Mohr-Coulomb constitutive model is classified as transitional state model. This behaviour is highlighted by the graph presented in Figure 2-10.

![Graph showing friction angle, cohesion, and tensile strength as functions of plastic strain](image)

**Figure 2-10:** Transitional state of FLAC3D models; modification of internal properties as a function of strain (Diederichs 2007).
It is important to recognize that although the CWFS approach can be applied to all types of brittle rockmasses, different rock types will present different strain limits for the onset of cohesion weakening and friction strengthening. These can be determined through a combination of laboratory testing and back analysis of excavations that have already undergone brittle damage.

A strain limit value of $1 \times 10^{-3}$ was used to mobilize the cohesion and fractional strength components of the Cobourg Limestone material in the FDM models. This value was chosen based on research published by Walton (2014).

### 2.3.4 Summary and Conclusion

Given an understanding of the differences in the DISL modelling approach and CWFS implementation, coupled with a simple examination of their behaviours presented in Figure 2-7 and Figure 2-10, respectively, the difference in the FEM and FDM modelling programs is apparent. While both systems utilize the same failure criterion with identical peak and residual values, the intermediate/transitional behaviour differs.

FEM models, such as RocScience’s RS$^2$ and RS$^3$, use two distinct curves to model brittle rockmass behaviour. As previously described, the DISL approach modifies the peak and residual FEM curves to better replicate the strain softening behaviour exhibited by brittle rocks. The transition between peak and residual cohesive and frictional strength components are effectively instantaneous within the model and cannot be user defined.

In contrast, FDM models such as FLAD$^{3D}$ incorporate a strain-dependency factor that allows the user to define the point at which the cohesive and frictional strength components are mobilized.
2.4 Cobourg Limestone Material Properties

2.4.1 Mechanical Properties

The following table (Table 2-2) depicts the material properties of the Cobourg Limestone sedimentary rock unit used in the modelling exercises. These values were based on the results of extensive testing carried out under numerous NWMO contracts (NWMO 2015). These tests include:

1. P- and S- wave velocity measurements.
2. Uniaxial compressive strength (UCS) testing (with acoustic emission monitoring).
3. Triaxial strength testing.
4. Brazilian tensile strength testing.
5. Direct shear strength testing of the bedding planes.
6. Long-term strength degradation testing.

The threshold values for CI and CD were obtained from the AE monitoring conducted during the UCS testing of the samples.

Table 2-2: Material properties for the Cobourg Limestone sedimentary rock unit (from Al et al. 2011).

<table>
<thead>
<tr>
<th>Material Properties</th>
<th>Sedimentary Rock Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (kg/m³)</td>
<td>2,700</td>
</tr>
<tr>
<td>Unconfined Compressive Strength (UCS) (MPa)</td>
<td>113</td>
</tr>
<tr>
<td>$m_t$ (UCS/T)</td>
<td>22</td>
</tr>
<tr>
<td>Tensile Strength (MPa)</td>
<td>5</td>
</tr>
<tr>
<td>Crack Initiation Stress (CI) (MPa)</td>
<td>45</td>
</tr>
<tr>
<td>Young’s Modulus (GPa)</td>
<td>42</td>
</tr>
<tr>
<td>Poisson’s Ratio</td>
<td>0.2</td>
</tr>
</tbody>
</table>
Also shown is an image of a sampled prepared for UCS testing (Figure 2-11). The sample has been equipped with extensometers and acoustic emissions tools (Jaczkowski et al. 2016).

Figure 2-11: Cobourg Limestone sample prepared for UCS testing at Queen’s University, equipped with extensometers and acoustic emissions tools (Jaczkowski et al. 2016).

2.5 Summary of Thesis Variables (Part II)

This chapter aims to summarize the thesis variables that will be examined based on the introductory information presented in Chapters 2.1 through 2.3. Each of the variables listed below will be examined on an interconnected level, with the aim of assessing both their individual and combined impact on EDZ development (magnitude, size, profile etc.) about underground nuclear waste storage excavations of varying shapes and dimensions.
2.5.1 Failure Criteria

The accuracy and applicability of input parameters when modelling tunnel excavations are paramount in ensuring reliable results are achieved. As such, this raises the question as to which failure criterion provides the most accurate results. This is again amplified given the fact that the modelling scope of this thesis is centered around Canada’s future high-level DGR, where ultimate certainty is vital to the long-term safety of the project.

International experts agree that the linear Mohr-Coulomb and non-linear Hoek-Brown failure envelopes serve as the most applicable failure criterion input when modelling geotechnical engineering designs in hard rock. Although these failure criteria are said to be “equivalent”, there are inherent issues that arise when transforming a linear failure envelope to a non-linear version.

As explained by Carter et al. (2008), although both failure criteria generate acceptable and relatively equivalent results (supported by Figure 2-12), it was observed that the Mohr-Coulomb failure criterion is better suited to rectangular openings given the fact it can more accurately reproduce expected tensile failure. Conversely, circular openings that generate less deterministic tensile failure can be better modelled using the Hoek-Brown failure criterion (Carter et al. 2008).

This theory was confirmed with numerical modelling of breakout observed at the URL test site in Pinawa, Manitoba (Hoek-Brown modelling) and pillars the McCreedy East Mine in Sudbury, Ontario (Hoek-Brown and Mohr-Coulomb).
Figure 2-12: FEM modelling of the two pillars in the McCreedy East Mine using Hoek-Brown (left) and equivalent Mohr-Coulomb (right) failure criteria. As shown, both failure criteria can replicate the damage profile (Carter et al. 2008).

The modelling results presented in Figure 2-12 also correlate to research published by Kaiser et al. (1996) in terms of the formation of a “baggage zone”. It is defined as the portion of un-yielded rock between the tunnel roof/floor or side walls and the concentrically yielded rockmass. This phenomenon is a result of the fact that rectangular tunnel profiles (or mine pillars as in Figure 2-12) rupture after CI is reached and move towards a more circular geometry. According to Kaiser et al. (1996), this baggage zone, while requiring support, also acts as an
energy absorbing barrier to dynamically yielding rockmass, and can serve to increase tunnel
safety if kept in place. It has also been referred to as “non-bulking dead weight” by Carter et al.
(2008).

This thesis will explore the issues that arise when modelling deep, brittle hard rock
tunnels of various geometries with both failure criteria, at varying stress states. To understand the
cause of the potential discrepancies in the modelling results presented in this thesis, an
understanding of each failure criterion is necessary. See Chapter 2.1 for a description of both the
generalized Hoek-Brown and equivalent Mohr-Coulomb failure criteria.

2.5.2 Numerical Modelling Software

2.5.2.1 Dimensionality (2D versus 3D Numerical Modelling)

RS\(^2\) uses a plane strain analysis wherein which the “two principal insitu stresses are in the
plane of the excavation and the third principal stress is out of plane” (RocScience 2016).
RocScience explains that since the excavation is of infinite length out-of-plane during a plane
strain analysis, the 3-dimensionality and its associated effects are not accounted for during the 2-
dimensional analysis. Additionally, RS\(^3\)’s 2-dimens does not account for shear stresses or strains
in the out-of-plane direction (i.e. in the direction of the tunnel axis). In a plane strain analysis, the
out-of-plane displacement (strain) is zero (RocScience 2016).

As such, a 2-dimensional modelling program such as RS\(^2\) does not provide insight into
the EDZ development about an advancing tunnel face. This data can however be obtained by
using RS\(^3\) and FLAC\(^{3D}\) since the 3-dimensionality of these programs does allow for the
computation of the out-of-plane displacements along the tunnel axis. The importance of this
behaviour is highlighted by the stress paths shown in Figure 2-13 on the following page; simply
put, plastic stresses are path dependent. Figure 2-13 displays the predicted damage resulting from
elastic stress paths near an advancing tunnel face (Diederichs et al. 2013).
Figure 2-13: Elastic stress paths in the tunnel crown and wall near the face of an advancing tunnel. Note that the east wall data is shown in the graph on the left, while the west wall data is shown on the right (from Diederichs et al. 2013).

When modelling a tunnel advance in RS$^2$, the 2-dimensional model staging uses a reduction in internal pressure to pseudo-simulate tunnel advance. In RS$^3$ and FLAC$^{3D}$, however, the true 3-dimensional excavation sequence is modelled with blast rounds simulated by using model slices in RS$^3$ and model depth (y-axis) in FLAC$^{3D}$. Using RS$^2$ therefore presents implications in terms of the accountability of stress rotation and the variability of the stress acting along the tunnel axis (i.e. out-of-plane/screen stress). This variation in modelling methodology is vital since it has also been proven that the degree of stress rotation can dramatically affect fracture propagation and subsequent damage development (again, refer to Figure 2-13). It is worth noting that the stress magnitudes about the tunnel perimeter are also directly affected by the model dimensions.

An understanding of the fact that RS$^2$ and RS$^3$/FLAC$^{3D}$ use two different methodologies (numerical solutions) to solve a 3-dimensional tunnel advance is paramount in explaining the anticipated discrepancies in the final results. Although each program uses identical material and strength properties and boundary conditions, the 2- and 3-dimensionalities of the programs will
affect the final yield profile and radial extent of EDZ development about the tunnel face and perimeter.

To understand modelling discrepancies with respect to EDZ development in brittle rock, a comparison between the results obtained using 2-dimensional and 3-dimensional models ($RS^2$ versus $RS^3$) has been conducted within the scope of this thesis.

### 2.5.2.2 The Use of Different Failure Criterion in Numerical Models

#### 2.5.2.2.1 FEM Modelling

Various authors have stated that FEM programs, such as those belonging to RocScience, are less sensitive to the use of different failure criteria. As shown in Figure 2-12, both criteria were able to replicate the damage profile exhibited in the McCreedy Mine Pillars (Carter et al. 2008).

However, a numerical modelling study was carried out by Fortsakis et al. (2011) on a 5 m diameter circular test tunnel with a variety of parameter combinations in order to compare the results obtained by using Hoek-Brown and equivalent Mohr-Coulomb failure criterion. Results of the FEM comparison study (using $RS^2$) showed that a $u_{ratio}$ (ratio of Hoek-Brown to Mohr-Coulomb displacement and yield results) diverged upon using “less favorable geotechnical conditions (decrease of rockmass quality or/and increase of overburden height”) (Fortsakis et al. 2011).

A case study was also carried out by Fortsakis et al. (2011) in relation to the Anthochori twin tunnels along the Egnatia Highway, location in Northern Greece. This 90 m deep tunnel was excavated in very weak siltstone (UCS of 5 MPa) with a local horizontal stress ratio ($K$) of 0.7. Again, the authors modelled the tunnel excavation using Hoek-Brown and equivalent Mohr-Coulomb failure criteria to enable a comparative assessment and quantification of the results.
Results of this FEM (RS$^2$) numerical modelling case study indicated that the Hoek-Brown numerical models generated more displacement and yield about the excavation (Figure 2-14).

Figure 2-14: (A): Deformation boundary, shown in terms of displacement, about the tunnel perimeter when modelling with Hoek-Brown and equivalent Mohr-Coulomb failure criterion. (B): Plastic yield zones generated when using Hoek-Brown (top) and equivalent Mohr-Coulomb (bottom) failure criteria (from Fortsakis et al 2011).

As shown by the results presented in Figure 2-14, the resulting rockmass displacement (shown in A) and yield profile (shown in B) are both larger when modelling with Hoek-brown failure criterion (Fortsakis et al 2011).

The results discussed in later chapters of this thesis will therefore aim to support these previously published conclusions in terms of the discrepancies in results achieved when modelling tunnel excavations with Hoek-Brown versus equivalent Mohr-Coulomb failure criteria.
2.5.2.2 FDM Modelling

Saiang et al. (2014) conducted a numerical modelling calibration experiment whereby true, observed results from an open pit mine site were compared to numerical models in FLAC$^{3D}$. It was observed that the Hoek-Brown model presented numerical instability upon excavation of the mine. It was also observed that the Hoek-Brown model presented very large strain, leading to displacement values that were nearly four times larger than those obtained from equivalent Mohr-Coulomb model, which better matched the true, insitu data. Upon modification of the Hoek-Brown model code (using Step rather than Solve), the results showed a stronger correlation. This phenomenon was attributed to the fact that the “Hoek-Brown model became immediately unstable when large plastic straining started to occur at or after yield. Due to of the lack of a plastic flow rule to relate stress and strain after yield, the resulting displacements and strains from Hoek-Brown becomes unpredictable and eventually produces results that are questionable” (Saiang et al. 2014). It was concluded that Mohr-Coulomb can however “relate stress and strain beyond failure, ultimately leading to results that are consistent and accurate” (Saiang et al. 2014).

Given this necessity to modify the FLAC$^{3D}$ code to try and achieve more reliable and accurate results, only the Mohr-Coulomb failure criterion will be used for the FDM FLAC$^{3D}$ modelling component of this thesis.

2.5.2.3 DISL in FEM Models versus CWFS in FDM Models

Chapters 2.2 and 2.3 provide an explanation of both the DISL approach to FEM numerical modelling of brittle rockmass behaviour, as well as the CWFS implementation in FDM models. Given the pre-developed understanding, as well as the difference and similarities between the two methods, this thesis will aim to determine the resulting discrepancies that arise when assessing equivalent FEM and FDM numerical models with DISL and CFWS approaches, respectively.
Chapter 3: Summary of Thesis Objectives and Organization

3.1 Thesis Objectives

By using numerous of 2-dimensional and 3-dimensional numerical models, the research presented in this thesis aimed to assess three primary objectives.

The foremost objective was to quantify the impacts of 2-dimensional macro-geometry choice details on resulting EDZ development about placement room and access tunnels. This included the study of tunnel shape, profile, corner details, blast round length and excavation methodology. Each of these variables were assessed in both 2-dimensinal and 3-dimensional FEM models (RS$^2$ and RS$^3$). Results of these studies have been presented in Chapter 4, Chapter 5, Chapter 6 and Chapter 8.

Secondly, a comparative study was also carried out in order to quantify the implications of using 2-dimensional versus 3-dimensional FEM numerical models on resulting EDZ development. RocScience RS$^2$ and RS$^3$ were used for this investigation. Chapter 7 presents the findings of this comparative study.

Finally, the choice of brittle damage model and its impact on EDZ development was also assessed and quantified. As previously explained, RS$^3$ and FLAC$^{3D}$ were used to implement and study the resulting EDZ that arises when using the DISL versus CWFS brittle damage approaches, respectively.

It is also worth highlighting that significant importance was also paid to the use of generalized Hoek-Brown versus equivalent Mohr-Coulomb failure criteria, the insitu stress regimes, as well as tunnel orientation with respect to the major and minor in-plane horizontal stresses.
The findings presented in this thesis will ultimately contribute to the determination of both safe and economic design details for future underground nuclear waste placement rooms and storage tunnels.

Note that a complete list of the numerical models used throughout this thesis can be found in Appendix A, along with verification and validation tools presented in Appendix B and Appendix C, respectively.

3.2 Outline of Thesis Results and Discussion

Chapters 4 through 9 contain the numerical modelling results and discussions pertaining to the various investigation presented herein and described below.

Chapter 4 explores the impacts of 2-dimensional macro-geometry choice details, such as tunnel shape and profile, on EDZ development. This investigation was carried out using 2D RS$^2$ FEM software.

Chapter 5 presents a 2-dimensional assessment of the impacts of failure criteria on the resulting EDZ development about DGR placement room tunnels. 2D RS$^2$ FEM models were assessed with both generalized Hoek-Brown DISL and equivalent Mohr-Coulomb DISL parameters.

Chapter 6 explores the impacts of corner details to further quantify the affects of 2-dimensional macro-geometry choice details on EDZ development. Again, 2D RS$^2$ FEM numerical modelling was used for this investigation.

Chapter 7 presents a comparative assessment of the impacts of 2-dimensional versus 3-dimensional numerical modelling on the resulting EDZ development. RS$^2$ and RS$^3$ software were used for the 2D and 3D FEM modelling investigations, respectively.

Chapter 8 serves to assess the impacts of various macro-geometry choice details on EDZ development within 3-dimensional numerical models. The impacts of tunnel shape, profile,
corner details, blast round length and excavation methodology were all quantified using 3D RS³ FEM numerical modelling software.

Chapter 9 presents a 3-dimensional assessment of the impacts of brittle damage model on EDZ development about DGR placement room tunnels. 3D RS³ FEM software was used to implement the DISL brittle damage approach, while 3D FLAC³D FDM software was used to implement the CWFS approach.

A summary of the key findings discussed throughout this thesis has been presented in Chapter 10, along with a discussion of potentially valuable future work.

A comprehensive list of the various numerical models used throughout this thesis, in addition to their input parameters, can be found in Appendix A. Appendix B describes the verification methods used on the numerical modelling software, while Appendix C describes the methods used to validate the setup and input parameters of the various numerical models.
Chapter 4: The Impacts of 2D Tunnel Shape and Profile Details on Brittle Damage Development and EDZ Categorization

4.1 Introduction

This chapter illustrates several key impacts of macro-geometry choices, such as shape and dimensions of placement rooms, as well as details associated with corner geometry, on the resulting EDZ development and baggage zone profile and their respective categorization.

4.1.1 Numerical Modelling: Input Parameters and Model Set-up

4.1.1.1 Material Properties and Strength Models

Refer to Chapter 2.4 for Cobourg Limestone material properties. Figure 2-6 in Chapter 2.3 displays the DISL input parameters for the Hoek-Brown and equivalent Mohr-Coulomb failure criteria.

4.1.1.2 Stresses and Depth

Each of the various models described in this chapter were assessed at a depth of 700 mbgs. Major and minor principal stresses were calculated based on two in-plane stress ratios \((K_{3,1})\) of 1.5 and 2.0. These aim to depict a suitable range within the Canadian context (Al et al. 2011). The following table (Table 4-1) depicts the various stress magnitudes for each of the \(K\) ratios. Note that for each of the models, the tunnel is aligned parallel to \(\sigma_3\), with \(\sigma_1\) acting perpendicular to the tunnel length, and \(\sigma_3\) thereby serving as the vertical stress.
Table 4-1: Principal stresses used for each of the models discussed in this chapter.

<table>
<thead>
<tr>
<th>Stresses (MPa)</th>
<th>$K_h = 1.5$</th>
<th>$K_{H} = 2.0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Major Principal Stress ($\sigma_1$)</td>
<td>28.4</td>
<td>37.8</td>
</tr>
<tr>
<td>Minor Principal Stress ($\sigma_3$)</td>
<td>18.9</td>
<td>18.9</td>
</tr>
<tr>
<td>Out-of-Plane Stress ($\sigma_2$)</td>
<td>23.6</td>
<td>28.4</td>
</tr>
</tbody>
</table>

4.1.1.3 Numerical Modelling Software

RocScience’s RS$^2$ version 9.0 was used for research presented in this chapter (see Chapter 2.2 for more information).

4.1.1.4 Mesh Discretization and Boundary Conditions

For each of the numerical models presented in this chapter, a square model boundary, 5x larger than the tunnel size, was used to reduce boundary effects. The external boundaries were all pinned since each of the models consist of tunnels located at a depth of 700 mbgs. The discretization density of the mesh elements was increased around the excavation to employ a pseudo-radially gradational model mesh, which aids to better delineate the extent of the EDZ around the tunnel perimeter. To properly capture brittle damage development and propagation around the excavations, the mesh element size was chosen in accordance with Walton and Diederichs (2015); the smallest mesh element measured no less than 3% of the tunnel radius.

4.1.1.5 Model Geometry

For the purposes of this investigation, various tunnel shapes were assessed to gather an understand of their impact on the EDZ propagation within a brittle rockmass at depth. The first of the three tunnel geometries consist of a rectangular tunnel geometry, 3.2 m in width, and 2.2 m in height (Figure 1-3), which replicates the dimensions of the Mark II placement rooms (Guo 2016). To explore the effects of rounded corners on the predicted decrease in EDZ propagation, the corners of the rectangular profile were rounded by 0.5 m (see Figure 4-1).
Finally, a horseshoe-shaped tunnel was also assessed; the dimensions of the horseshoe tunnel were kept similar to the Mark II rectangular placement room tunnel design (Guo 2016) in order to draw comparative conclusions in terms of EDZ propagation.

Figure 4-1: Trial excavation dimensions with (left) rounded corners and (right) arched roof.

4.2 Results and Discussion

4.2.1 The Impacts of 2D Tunnel Shape and Profile Details on Brittle Damage Development

As anticipated, a larger K ratio resulted in a greater EDZ development, regardless of the tunnel profile. This is highlighted by the results presented in Figure 4-2.
Figure 4-2: Comparison of EDZ/HDZ in the roof of an arched placement room tunnel for $K_{h,H}$ of 1.5 and 2.0 (H-B DISL parameters).

To assess the impact of the shape effects (corner geometries) on tunnel yield, the extent of yielded elements (width and height of yielded zone about each of the excavation geometries) was assessed, as summarized in Table 4-2 and Table 4-3 (also shown in Figure 4-3 for a $K_{h}$ of 2.0) and discussed in the following sections.

Table 4-2: Extent of yield (from center line) with a $K_{h}$ of 1.5.

<table>
<thead>
<tr>
<th>Geometry</th>
<th>Yield</th>
<th>Mohr-Coulomb</th>
<th>Hoek-Brown</th>
<th>Ratio (H-B: M-C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rectangle</td>
<td>Width (m)</td>
<td>0.87</td>
<td>1.73</td>
<td>1.99</td>
</tr>
<tr>
<td></td>
<td>Height (m)</td>
<td>0.45</td>
<td>1.05</td>
<td>2.33</td>
</tr>
<tr>
<td>Rounded Corners</td>
<td>Width (m)</td>
<td>0.75</td>
<td>1.58</td>
<td>2.11</td>
</tr>
<tr>
<td></td>
<td>Height (m)</td>
<td>0.36</td>
<td>1.04</td>
<td>2.89</td>
</tr>
<tr>
<td>Horseshoe</td>
<td>Width (m)</td>
<td>0.53</td>
<td>1.68</td>
<td>3.17</td>
</tr>
<tr>
<td></td>
<td>Height (m)</td>
<td>0.30</td>
<td>0.83</td>
<td>2.77</td>
</tr>
</tbody>
</table>
Table 4-3: Extent of yield (from center line) with a $K_H$ of 2.0.

<table>
<thead>
<tr>
<th>Geometry</th>
<th>Yield</th>
<th>Mohr-Coulomb</th>
<th>Hoek-Brown</th>
<th>Ratio (H-B:M-C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rectangle</td>
<td>Width (m)</td>
<td>1.68</td>
<td>1.69</td>
<td>1.01</td>
</tr>
<tr>
<td></td>
<td>Height (m)</td>
<td>1.29</td>
<td>1.18</td>
<td>0.91</td>
</tr>
<tr>
<td>Rounded Corners</td>
<td>Width (m)</td>
<td>1.56</td>
<td>1.56</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>Height (m)</td>
<td>1.09</td>
<td>1.13</td>
<td>1.04</td>
</tr>
<tr>
<td>Horseshoe</td>
<td>Width (m)</td>
<td>1.70</td>
<td>1.70</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>Height (m)</td>
<td>1.23</td>
<td>1.04</td>
<td>0.85</td>
</tr>
</tbody>
</table>

Note that the sharp and rounded rectangular profiles result in similar roof and floor damage zones. The increased rounding in the crown of the horseshoe tunnel profile is noted to result in a far more dramatic decrease in yield, as compared to mere corner rounding; this conclusion holds true for both stress regimes.

Figure 4-3: Yield and shear strain results from Table 4-3 (obtained using RS2).

One striking aspect of Figure 4-3 is the well-defined block created above the two rectangular profiles and in the floor of all the tunnels (compared to the continuous yielded zone above the roof in the arched case). This creation of a curved yield zone encompassing a
minimally damaged block of rock is known as “the baggage zone” in brittle rock engineering, as coined by Kaiser et al. (1996). These authors explain that “stress-induced fractures propagate from stress raiser at the corners of the excavations forming semi-circular fracture patterns (onion skinning) connecting zones of high stress” (Kaiser et al. 1996). The baggage zone is the portion of un-yielded rock between the tunnel roof/floor and the concentrically yielded rockmass within the tensile fracture zone. This phenomenon is a result of the fact that rectangular tunnels rupture after CI is reached and move towards a more circular geometry. According to Kaiser et al. (2006), this baggage zone, while requiring support, also acts as an energy absorbing barrier to dynamically yielding rockmass, and can serve to increase tunnel safety if kept in place. The authors have observed such baggage zones in mining tunnels.

Kaiser et al. (1996) stated that the depth/extent of baggage is roughly 20% of the width or height of the tunnel geometry. Given the size of the baggage zones observed in the various models (and presented in the Figure 4-3), 2D numerical modelling results presented in this chapter correlated to this published conclusion.

In general, the rounding of the right-angle corners acts to reduce the overall yield by 1 – 20%. The greater value of height yield ratio is the result of the $K_h$ value of 1.5, which generates increased yield orthogonal to $\sigma_1$, thereby having a greater impact on the height of yielded rockmass.

4.2.1.1 Hoek-Brown DISL versus Equivalent Mohr-Coulomb DISL

The impact of using equivalent Mohr-Coulomb or generalized Hoek-Brown failure criterion at different (increasing) stress regimes were also explored. For staged excavations simulated in RS², aforementioned results prove that a lower stress regime is far more sensitive to the use of different failure criteria. It was discovered that for the case of a $K_h$ of 1.5, the tunnels modelled with generalized Hoek-Brown failure criterion present approximately double (to triple)
the amount of yield about the tunnel corners, as compared to tunnels assessed with equivalent Mohr-Coulomb parameters. A graphical representation of these results can also be seen in Figure 4-4. The effects of using equivalent failure criterion will be further discussed in the following section.

![Graphical representation of the EDZ development for each of the three geometries. The dependence on the failure criterion for a \( K_h \) of 1.5 is well highlighted. Lines for Rectangle M-C and Rounded M-C are overlain by Horseshoe M-C.](image)

Figure 4-4: Graphical representation of the EDZ development for each of the three geometries. The dependence on the failure criterion for a \( K_h \) of 1.5 is well highlighted. Lines for Rectangle M-C and Rounded M-C are overlain by Horseshoe M-C.

When modelling the various geometries with a \( K_h \) of 1.5 and Mohr-Coulomb failure criterion, the rectangular tunnel, as well as the tunnel with rounded corners, presented no EDZ development above the roof or within the floor; damage was limited to the corners (see Figure 4-5 and Figure 4-6 on the following page).
Conversely, these geometries experienced concentric yield about the roof and floor when modelled with generalized Hoek-Brown failure criterion (while still using a $K_h$ of 1.5), generating a baggage zone (see Figure 4-5).

**Figure 4-5:** Left: Localized fracturing in a vertically loaded rock pillar encompassing a minimally fractured baggage zone. Top Right: Incomplete damage zone initiation in tunnel roof generated using equivalent Mohr-Coulomb DISL failure criterion. Bottom Right: Clearly defined baggage zone generated upon using Hoek-Brown DISL failure criterion. Both modelled with a $K_h$ of 1.5 and a depth of 700 mbgs.

When using an increased $K_h$ of 2.0, the tunnel geometries presented nearly identical results when modelled with either the Mohr-Coulomb or equivalent generalized Hoek-Brown failure criteria. Unlike results obtained when using a $K_h$ of 1.5, a baggage zone occurred when modelling with either of the two failure criteria at a $K_h$ of 2.0. This can be supported by the data presented in Table 4-2 and Table 4-3.

It is worth noting that in correlation to conclusions published by Kaiser et al. (1996), the depth of the baggage zones was roughly 20% of the tunnel height for both $K$ ratio stress regimes.
For both cases, the yield zone around the roof of the horseshoe tunnel was relatively insensitive to the yield criterion selected (denoted by the grouping of the green lines in Figure 4-4 and Figure 4-6).

Figure 4-6 graphically highlights the data presented in Table 4-3; it can therefore be concluded that the dependence on the failure criteria is negligible when assessing the stability of a tunnel with an increased $K_H$ ratio of 2.0.

**Figure 4-6:** Graphical representation of the EDZ development for each of the three geometries with a $K_H$ of 2.0. Note the diminished dependence on failure criterion as compared to the results presented in Figure 4-4. Line for Horseshoe H-B is overlain by Horseshoe M-C.

The following Chapter 5 will aim to delineate the stress state at which both failure criteria produce similar results.
4.2.2 The Impacts of 2D Tunnel Shape and Profile Details on EDZ Categorization

Perras and Diederichs (2016) have developed a system for delineating the extent of the EDZ regions. The following figure (Figure 4-7) depicts the study in which the EDZ development around a brittle circular tunnel was categorized based on results obtained from numerical modelling of a limestone. It has been proven that principal stresses ($\sigma_1$ and $\sigma_3$), yielded elements, volumetric strain and maximum shear strain aid in delineating the EDZ regions (Perras and Diederichs 2016). These parameters were all measured radially from the tunnel surface, aligned with $\sigma_3$ (direction of most damage). The HDZ/EDZ$_i$ boundary is defined by the onset of positive $\sigma_3$, while the EDZ$/EDZ_o$ boundary is attributed to the reversal in the volumetric strain (lowest value). Finally, the extent of plastic yield (yielded elements) serves as the $EDZ_o$ extent.

Figure 4-7: EDZ development around a circular tunnel; results pertain to a brittle limestone (from Perras and Diederichs 2016).
Modelling results using Hoek-Brown failure criterion, with a $K_H$ of 2.0, as depicted in the following figures Figure 4-8 and Figure 4-9, suggest that the guidelines of Diederichs and Perras (2016) are not entirely appropriate for non-curved geometries.

Figure 4-8: EDZ development for a rectangular tunnel using Hoek-Brown failure criterion. Note the highlight of the baggage zone. Inset are figures corresponding to $\sigma_3$ (left) and volumetric stain (right).

In correlation to Kaiser et al. (1996), these baggage zones have been confirmed to measure roughly 20% of the tunnel height and are independent of failure criteria. It is important to recall that this baggage zone, while requiring support, can also act as an energy absorbing barrier to dynamically yielding rockmass, and can serve to increase tunnel safety if kept in place.
This is relevant to the overarching topic of long-term safety of the DGR throughout each stage of the project lifespan.

Figure 4-9: EDZ development for a rectangular tunnel using Hoek-Brown failure criterion. Note the highlight of the $\sigma_3$ reversals. Inset are figures corresponding to $\sigma_3$ (left) and volumetric stain (right).

4.3 Conclusions

This study highlights several issues and challenges encountered when modelling brittle EDZ evolution with a 2D FEM continuum model. The choice of approximation for the DISL brittle damage model (Hoek-Brown or Mohr-Coulomb) impacts the nature and extent of EDZ for moderate stress states.
Corners and flat boundaries create baggage zones entraining undamaged rockmass that have been observed in mining tunnels. The existence of the baggage zone raises the question as to its inclusion within the HDZ when categorizing EDZ for nuclear waste disposal purposes.

Secondly, there are issues when trying to delineate the HDZ/EDZ₁ and EDZ₀/EDZᵋ boundaries since the onset of positive σ₃ has been noted to occur after the volumetric strain reversal. Furthermore, multiple reversals in positive-to-negative σ₃ are noted to occur within the yielded rockmass due to this baggage.

Overall, the generalized guidelines for model interpretation need to be revised to encompass corner and baggage issues.
Chapter 5: A 2-Dimensional Assessment of the Impacts of Failure Criteria on Brittle Damage Development

5.1 Introduction

The following chapter will aim to assess the impacts of failure criteria selection (Hoek-Brown and equivalent Mohr-Coulomb) on modelling of brittle damage development around deep, underground excavations. Based on the previous understanding developed in Chapter 4.2.1.1 regarding the discrepancies that arise when modelling with different failure criteria at varying stress states, this chapter aims to delineate the stress states at which Hoek-Brown and equivalent Mohr-Coulomb models generate comparable results. This will be assessed in terms of EDZ development, baggage zone development and profile, as well as the failure mechanism.

5.1.1 Failure Criteria

The accuracy and applicability of input parameters when modelling tunnel excavations are paramount in ensuring reliable results are achieved. As such, this raises the question as to which failure criterion provides the most accurate results. International experts agree that the linear Mohr-Coulomb and non-linear Hoek-Brown failure envelopes serve as the most applicable failure criteria input when modelling geotechnical engineering designs in hard rock. Although these failure criteria are said to be “equivalent” when co-calibrated, there are inherent issues that arise when transforming a linear failure envelope to a non-linear version. This chapter will explore the issues that arise when modelling deep, brittle hard-rock tunnels of various geometries with both failure criteria, at varying stress states.

To understand the cause of the potential discrepancies in the results present in the following sections, an understanding of each failure criterion is necessary. The generalized Hoek-Brown and equivalent Mohr-Coulomb failure criteria are briefly described in the Chapter 2.1.
Recall that Chapter 2 discusses the detailed process and tools used to obtain the generalized Hoek-Brown failure criterion, as well as the equivalent Mohr-Coulomb failure criterion for the brittle rockmass under study.

5.1.2 Numerical Modelling: Input Parameters and Model set-up

5.1.2.1 Material Properties and Strength Models

Refer to Chapter 2.4 for Cobourg Limestone material properties. Figure 2-6 in Chapter 2.3 displays the DISL input parameters for the Hoek-Brown and equivalent Mohr-Coulomb failure criteria.

5.1.2.2 Stresses and Depth

Each of the various models described in this chapter were assessed at a depth of 700 mbgs. Major and minor principal stresses were calculated based on six in-plane stress ratios (K) ranging from 1.5 to 2.0. The following table (Table 5-1) depicts the various stress regimes for each of the K ratios. Note that for each of the models, the tunnel is aligned parallel to $\sigma_2$, with $\sigma_1$ acting perpendicular to the tunnel length, and $\sigma_3$ thereby serving as the vertical stress.

Table 5-1: Principal stresses used for each of the models discussed in this chapter.

<table>
<thead>
<tr>
<th></th>
<th>Major Principal Stress ($\sigma_1$)</th>
<th>Minor Principal Stress ($\sigma_3$)</th>
<th>Out-of-Plane Stress ($\sigma_2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>K = 1.5</td>
<td>28.4</td>
<td>18.9</td>
<td>23.6</td>
</tr>
<tr>
<td>K = 1.6</td>
<td>30.2</td>
<td>18.9</td>
<td>24.6</td>
</tr>
<tr>
<td>K = 1.7</td>
<td>32.1</td>
<td>18.9</td>
<td>25.5</td>
</tr>
<tr>
<td>K = 1.8</td>
<td>34.0</td>
<td>18.9</td>
<td>26.5</td>
</tr>
<tr>
<td>K = 1.9</td>
<td>35.9</td>
<td>18.9</td>
<td>27.4</td>
</tr>
<tr>
<td>K = 2.0</td>
<td>37.8</td>
<td>18.9</td>
<td>28.4</td>
</tr>
</tbody>
</table>
5.1.2.3 Numerical Modelling Software

For this investigation, RS³ version 9.0 was utilized (see Chapter 2.2 for more information). See Appendix A for a complete list of models presented in Chapter 5.

5.1.2.4 Mesh Discretization and Boundary Conditions

For each of the numerical models presented in this chapter, a square model boundary, 5x larger than the tunnel size, was once again used to reduce boundary effects. The external boundaries were all pinned and the mesh element discretization density was increased around the excavation to employ a pseudo-radially gradational model mesh. As discussed in Chapter 3, the smallest mesh element measured no less than 3% of the tunnel radius (Walton and Diederichs 2015).

5.1.2.5 Model Geometry

For the purposes of this investigation, various tunnel shapes were assessed to gather an understanding of their impact on the EDZ propagation within a brittle rockmass at depth. These geometries include a square with sharp and rounded corners, a rectangle with sharp and rounded corners, a square- and rectangular-based horseshoe profile, as well as a circular geometry. The dimensions of each of the seven geometries have been displayed in Figure 5-1 to Figure 5-4.

Similar to the geometries used in Chapter 3, the first two of the seven tunnel geometries consist of square-shaped profiles; the first presents dimensions of 3.2 m in width and height, while the second consists of 0.5 m rounded corners (see Figure 5-1).
The rectangular geometries measure 3.2 m in width and 2.2 m in height; the corners are again rounded by 0.5 m on the second rectangular model (dimensions are highlight in Figure 5-1). These dimensions were selected in order to replicate the shape of the Mark II placement rooms (as shown in Figure 1-3).
The horseshoe-shaped tunnels present both a square and rectangular-base profile, measuring 3.2 m and 2.2 m in height, respectively. These can both be seen below in Figure 5-3.

![Figure 5-3: Dimensions of the horseshoe tunnel geometries.](image)

Finally, the circular tunnel depicted in Figure 5-4 below, presents a diameter of 3.2 m for consistency.

![Figure 5-4: Dimensions of the circular tunnel profile.](image)
5.2 Results and Discussion

5.2.1 The Impacts of Failure Criteria and Insitu Stress on EDZ Development

As previously understood (refer to Chapter 4.2.1.1) the choice of approximation for the DISL brittle damage model (generalized Hoek-Brown or equivalent Mohr-Coulomb) impacts the nature and extent of EDZ and baggage zone development for low-to-moderate stress states ($K_h$ of 1.5). The dependency on the failure criteria becomes negligible at a $K_h$ of 2.0.

Each of the seven aforementioned tunnel geometries were modelled using both failure criteria at a $K$ ranging from 1.5 to 2.0 in order to delineate the stress state at which the modelling results of each geometry becomes independent of the DISL failure criteria. The yield was assessed both in terms of radial height and width propagation, as well as the generation of the baggage zone.

Figure 5-5 on the following page depicts the method used to assess the width and height of yield about the excavation boundary. The four width measurements, taken from the centerline of the excavation, were averaged. This same method was also used to determine the height of yielded elements, representing the extent of the EDZ (EDZe and EIZ boundary). Note that only regions of “100% yield” were measured as part of the yield zone; this was conducted to enable comparisons between later results obtained from RS$^3$ numerical modelling, which is only capable of illustrating regions of 100% yield (rather than the range in percentage that can be seen in RS$^2$).
5.2.1.1 Width of Radial Yield Propagation

To quantify the extent of EDZ propagation about each of the geometries at varying stress states, the width of the yielded elements (measured as shown in Figure 5-5), was measured. This was then tabulated, with the results presented in Figure 5-6; note that solid and dashed lines are used to distinguish between the results corresponding to the generalized Hoek-Brown and Mohr-Coulomb models, respectively. Upon examining the results presented in the graph (Figure 5-6), it is apparent that there is a strong relationship between the horizontal stress ratio at which both failure criteria generate similar results in terms of the measured width of yielded elements; Table 5-2 presents this tabulate data.
Based on the results presented in Figure 5-6, it is apparent that lower insitu stress states result in discrepancies in modelling with different failure criteria; a larger EDZ is observed in the Hoek-Brown models as compared to the equivalent Mohr-Coulomb models. Table 5-2 serves to compile the horizontal stress ratios at which the width of yielded elements in the Hoek-Brown models surpass those of the equivalent Mohr-Coulomb models.

**Figure 5-6:** This graph depicts the relationship between the width of yielded elements about the corners of various tunnel geometries when using generalized Hoek-Brown and equivalent Mohr-Coulomb failure criterion.
It should be noted that up until the point at which both failure criteria generate similar results, the Mohr-Coulomb models generate less yield than the generalized Hoek-Brown counterparts (see Figure 5-6).

**Table 5-2**: This table displays the tabulated results of the data presented in Figure 5-6. The right-most column lists the stress state at which the number of yielded elements (width) generated using the equivalent Mohr-Coulomb failure criterion becomes equal to those obtained from the Hoek-Brown models (for each tunnel geometry).

<table>
<thead>
<tr>
<th>Tunnel Profile</th>
<th>K</th>
</tr>
</thead>
<tbody>
<tr>
<td>Square</td>
<td>1.60</td>
</tr>
<tr>
<td>Square (Rounded Corners)</td>
<td>1.60</td>
</tr>
<tr>
<td>Rectangle</td>
<td>1.70</td>
</tr>
<tr>
<td>Rectangle (Rounded Corners)</td>
<td>1.70</td>
</tr>
<tr>
<td>Square-base Horseshoe</td>
<td>1.75</td>
</tr>
<tr>
<td>Rectangular-base Horseshoe</td>
<td>1.80</td>
</tr>
<tr>
<td>Circle</td>
<td>Nearly equal at all stress regimes</td>
</tr>
</tbody>
</table>

**5.2.1.2 Height of Radial Yield Propagation**

As previously mentioned, the height of radial EDZ development about the excavation boundary was also assessed. This was conducted in addition to the aforementioned width results in order to gather complete results of EDZ development around different tunnel geometries at varying stress states. Figure 5-5 highlights the methodology by which the height of yielded elements was measured.
Figure 5-7: This graph depicts the relationship between the height of yielded elements about the corners of various tunnel geometries when using generalized Hoek-Brown and equivalent Mohr-Coulomb failure criterion.

Upon examining the data displayed in Figure 5-7, it is again apparent that for lower stress states (lower K ratio), the modelling results obtained using the equivalent Mohr-Coulomb failure criterion present less EDZ development than those produced by the Hoek-Brown models.

Table 5-3 presents the horizontal stress ratios at which the height of yielded elements in the Hoek-Brown models are surpassed by those of the equivalent Mohr-Coulomb models.
Table 5-3: This table displays the tabulated results of the data presented in Figure 5-7. The right-most column lists the stress state at which the number of yielded elements (height) generating using the equivalent Mohr-Coulomb failure criterion first exceeds to those obtained from the Hoek-Brown models (for each tunnel geometry).

<table>
<thead>
<tr>
<th>Mohr-Coulomb Yield Exceeds Hoek-Brown (Height)</th>
<th>K</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tunnel Profile</td>
<td>K</td>
</tr>
<tr>
<td>Square</td>
<td>1.65</td>
</tr>
<tr>
<td>Square (Rounded Corners)</td>
<td>1.55</td>
</tr>
<tr>
<td>Rectangle</td>
<td>1.65</td>
</tr>
<tr>
<td>Rectangle (Rounded Corners)</td>
<td>1.65</td>
</tr>
<tr>
<td>Square-base Horseshoe</td>
<td>1.80</td>
</tr>
<tr>
<td>Rectangular-base Horseshoe</td>
<td>1.75</td>
</tr>
<tr>
<td>Circle</td>
<td>Nearly equal at all stress regimes</td>
</tr>
</tbody>
</table>

In spite of the fact that the aforementioned results do present numerous similarities, it is important to acknowledge the following slight difference. When assessing both the width and height of yielded elements, it was observed that at lower stress regimes, the results generated using the generalized Hoek-Brown failure criterion were higher than those pertaining to the equivalent Mohr-Coulomb models.

Upon reaching a similar intermediate stress state (see Table 5-2 and Table 5-3), the Mohr-Coulomb models presents more yield (rather than the same amount) than the equivalent Hoek-Brown counterpart models when measuring the height of yield as compared to the width of yielded elements. This might therefore suggest that the height of yielded elements is more susceptible to discrepancies when modelling with different failure criterion.

5.2.1.3 Failure Criteria and Tunnel Shape/Profile Relationship

Based on the findings presented in Table 5-2 and Table 5-3, it can be concluded that the horizontal stress ratio at which the width of yielded element becomes equal for both failure criteria is related the tunnel dimension ratio; that is, the ratio of tunnel width to height ($u_{wb}$).
Tunnel profiles with a $u_{wh}$ closer to 1 (i.e. circular or square tunnel geometries) appear to be far less sensitive to the horizontal stress ratio when modelling with different failure criteria. This is supported by the fact that both square tunnel geometries presented similar yield results from a K of 1.6 upwards, while the circular tunnel presented nearly identical yield results regardless of the horizontal stress ratio or failure criteria (see continuous tight grouping in green “circle” lines in Figure 5-6 and Figure 5-7). When assessing the results of the rectangular and horseshoe tunnel profiles, the horizontal stress ratio at which the yield resulting from using generalized Hoek-Brown and Mohr-Coulomb is nearly identical is slightly higher, occurring at a K of 1.7 to 1.8.

In terms of the height of yielded elements about the excavation, it is again apparent that the tunnel geometries with a $u_{wh}$ closer to 1 appear to be far less sensitive to the horizontal stress ratio when modelling with different failure criteria. This can be again supported by the fact that the two failure criteria produce similar results with the lowest average stress regime (1.61) for the square-shaped tunnels. Furthermore, the circular tunnel presented nearly identical yield results regardless of the horizontal stress ratio and failure criteria; this is made evident by the tight grouping of the green “circle” lines in Figure 5-6 and Figure 5-7. For the results pertaining to the rectangular- and horseshoe-shaped tunnel profiles, Figure 5-6 and Figure 5-7 highlight the consistency with the aforementioned results. These tunnel profiles once again appear to be far more sensitive to the use of different failure criterion when assessing the width of yielded elements. The horizontal stress ratios at which the yield resulting from using equivalent Mohr-Coulomb exceeds the Hoek-Brown results is yet again higher than that of the square- and circular-shaped tunnel profiles; the cross-overs occur at average K values of 1.66 to 1.79, for the rectangular- and horseshoe-shaped tunnels, respectively.
5.2.2 The Impacts of Failure Criteria on the Baggage Zone Development and Profile

5.2.2.1 Baggage Zone Development

As previously described in Chapter 4.2.1.1, it is understood that the use of varying failure criteria plays a dramatic role in the shape and extent of EDZ development. It is understood that for square and rectangular tunnel geometries, a well-defined block is created above the roof and within the floor of a tunnel. This creation of a curved yield zone encompassing a minimally damaged block of rock is known as the baggage zone in brittle rock engineering.

The use of varying failure criteria was also proven to be affected by the insitu stress regime. With reference to the results presented in Chapter 4.2.1.1, when modelling the various geometries with a $K_h$ of 1.5, the Mohr-Coulomb rectangular models presented no EDZ development or baggage zone above the roof or within the floor; damage was limited to the corners. Conversely, models experienced concentric yield about the roof and floor when modelled with generalized Hoek-Brown failure criterion, generating a baggage zone. See Figure 5-8 on the following pages for proof of this phenomenon.
Figure 5-8: Comparison of resulting shear strain profiles and yielded elements when using Mohr-Coulomb versus Hoek-Brown failure criteria at a low insitu stress regime.
When using an increased stress regime ($K_H$ of 2.0), the tunnel geometries presented nearly identical results when modelled with either the Mohr-Coulomb or equivalent generalized Hoek-Brown failure criteria. As shown in Figure 5-9 on the following page, a baggage zone occurred when modelling with either of the two failure criteria at a $K_H$ of 2.0.

Recall that for both insitu stress regimes, the yield zone around the roof of the horseshoe tunnel geometries was relatively insensitive to the yield criterion and does not generate a baggage zone.

Based on the results presented in this chapter, it has been proven that once the horizontal stress ratio reaches and surpasses a value of roughly 1.75, the EDZ development about an underground opening, of any shape, becomes independent of the failure criterion. At this insitu stress regime, the baggage zone generated by both failure criterion becomes nearly identical in size. As per Martin et al. 1996, the depth of the baggage zone in the crown and floor was also observed to measure roughly 20% of the tunnel height.
Figure 5-9: Comparison of resulting shear strain profiles and yielded elements when using Mohr-Coulomb versus Hoek-Brown failure criteria at a high insitu stress regime.
5.2.2.2 Baggage Zone Profile/Shape

Although the FEM numerical modelling results pertaining to the generation of baggage zones do present a strong relationship at high stress regimes (K = 1.75 and greater) when modelled with either failure criteria, it is still worth noting that the profile/shape of the baggage zone varies with failure criteria.

As per the results presented in Figure 5-8 and Figure 5-9, the shape of the overall EDZ and baggage zone is observed to be sharper/more angular when using the linear Mohr-Coulomb failure criterion. The non-linear Hoek-Brown models generally result in a baggage zone with slightly more rounded (less angular) edges. While the height of the resulting baggage zones are still noted to measure roughly 20% of the tunnel height, this variation thereby manifests as a slightly smaller baggage zone volume in the Hoek-Brown models as compared to the equivalent Mohr-Coulomb models.

The following Figure 5-10, depicting the rectangular tunnel with rounded corners, serves to best highlight these minor discrepancies in baggage zone profile.
Figure 5-10: Comparison of the shape and volume of the baggage zone (assessed in terms of resulting shear strain and yielded elements) generated when using Mohr-Coulomb versus Hoek-Brown failure criteria.

5.2.3 The Impacts of Failure Criteria on Resulting Failure Mechanism

The results presented in Figure 5-8 and Figure 5-9 present a clear depiction of the variability in failure mechanism that occurs in the Hoek-Brown versus the equivalent Mohr-Coulomb models. With reference to the legends on the left side of the figures, it is evident that the Hoek-Brown models present a larger degree of tensile failure; this can be observed both along the
excavation boundary, as well as within the yield surface (denoted by the concentric region of increased maximum shear strain about the crown and floor).

It is also worth noting that the magnitude of maximum shear strain is generally 0.5 – 1 orders of magnitude larger in Mohr-Coulomb models.

Both traits can be easily seen in the results of Figure 5-10 on the previous page.

5.3 Conclusions

Figure 5-11 serves as a normalized representation of the results presented in Figure 5-6 and Figure 5-7. The general trend in the data presented in Figure 5-11 further supports the conclusions that tunnel geometries with a $u_{\text{wb}}$ closer to 1 appear to be far less sensitive to the horizontal stress ratio when modelling with either generalized Hoek-Brown or equivalent Mohr-Coulomb failure criteria.
Figure 5-11: Normalization of the results presented in Figure 5-6 and Figure 5-7 Note that “w” denotes width measurements and “h” denotes height measurements.

By implementing the normalized y-axis, this conclusion is again highlighted by the fact that the circular- and square-shaped tunnel geometries present yield ratios of approximately 1, regardless of the horizontal stress ratio. Furthermore, it is evident that as the tunnel profile diverges from a 1:1 \( u_{wh} \) (and becomes more rectangular), the model results vary significantly depending on the use of different failure criteria at lower stress states.

Based on the results presented in this chapter, it can also be concluded that as the horizontal stress ratio reaches and surpasses roughly 1.75, the EDZ development about an underground opening, of any shape, becomes independent of the failure criterion.
The existence and profile of the baggage has been proven to be dependent on both the stress regime and the failure criterion. Below a $K$ of 1.75, a baggage zone does not present itself in the Mohr-Coulomb models, while the use of different failure criteria at higher stress regimes results in nearly identical baggage zone formation. The use of Mohr-Coulomb failure criterion results in the formation of a more angular baggage zone in the crown and floor of the various tunnel geometries. Nevertheless, the depth of the baggage zone in the crown and floor was still observed to measure roughly 20% of the tunnel height.

In relation to the research conducted by Fortsakis et al. (2011) (at a $K$ of 0.7), the results of this FEM ($RS^2$) numerical modelling study proved that at low insitu stress regimes, Hoek-Brown numerical models generated more yield about the excavation boundary as compared to equivalent Mohr-Coulomb models. As the insitu stress regimes surpass a horizontal stress ratio of approximately 1.75, the resulting failure zone becomes relatively independent of the failure criteria. This correlates to research published by Carter et al. (2008), whereby a case study at the McCriddy East Mine in Sudbury, Ontario proved that both failure criteria can replicate the spalling damage observed in mine pillars as a result of high insitu stresses (refer to Figure 2-12).

Finally, the Hoek-Brown models were noted to present more tensile failure both along the excavation boundary and within the yield surface (denoted by the concentric region of increased maximum shear strain about the crown and floor). However, the magnitude of maximum shear stain was generally observed to be roughly $0.5 - 1$ orders of magnitude larger in Mohr-Coulomb models.
Chapter 6: A 2-Dimensional Assessment of The Impacts of Tunnel Corner Geometry on Brittle Damage Development

6.1 Introduction

In this Chapter, we will aim to assess the impact of tunnel corner geometry on 2D FEM modelling of brittle damage development around deep, underground excavations. The impact of rounded corners on EDZ development has been assessed by using square and circular tunnel profiles as baseline worst- and best-case scenarios for the modelling exercise, respectively. The degree of rounding applied to the corners of each tunnel profile has been described in later sections.

Since it is understood that failure criteria can impact the resulting brittle damage and EDZ development, each tunnel profile has been modelled using both generalized Hoek-Brown and equivalent Mohr-Coulomb failure criteria. Please refer to Chapter 2.1 for a description of both failure criteria. To provide consistency with previous chapters, varying stress regimes will be assessed to determine their relationship to corner geometry.

6.1.1 Numerical Modelling: Input Parameters and Model set-up

6.1.1.1 Material and Strength Properties

Refer to Chapter 2.4 for Cobourg Limestone material properties. Figure 2-6 in Chapter 2.3 displays the DISL input parameters for the Hoek-Brown and equivalent Mohr-Coulomb failure criteria.

6.1.1.2 Stresses and Depth

Each of the various models described later in this chapter were assessed at a depth of 700 mbgs. Major and minor principal stresses were calculated based on horizontal stress ratios (K) of 1.5, 1.75, 2.0 and 2.5. The following Table 6-1 depicts the various stress regimes for each of the
K ratios. Note that for each of the models, the tunnel was aligned with \( \sigma_2 \), with \( \sigma_1 \) acting perpendicular to the tunnel length, and \( \sigma_3 \) thereby serving as the vertical stress.

Table 6-1: Principal stresses used for each of the models discussed in this chapter.

<table>
<thead>
<tr>
<th>K</th>
<th>Major Principal Stress (( \sigma_1 ))</th>
<th>Minor Principal Stress (( \sigma_3 ))</th>
<th>Out-of-Plane Stress (( \sigma_2 ))</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5</td>
<td>28.4</td>
<td>18.9</td>
<td>23.6</td>
</tr>
<tr>
<td>1.75</td>
<td>33.1</td>
<td>18.9</td>
<td>26.0</td>
</tr>
<tr>
<td>2.0</td>
<td>37.8</td>
<td>18.9</td>
<td>28.4</td>
</tr>
<tr>
<td>2.5</td>
<td>47.3</td>
<td>18.9</td>
<td>33.1</td>
</tr>
</tbody>
</table>

6.1.1.2.1 Numerical Modelling Software

RS\(^2\) version 9.0 was utilized for each of the models present in this chapter (see Chapter 2.2 for more information). See Appendix A for a complete list of models presented in Chapter 6.

6.1.1.2.2 Mesh Discretization and Boundary Conditions

For each of the numerical models presented in this chapter, a square model boundary, 5x larger than the tunnel size, was once again used to reduce boundary effects. The external boundaries were all pinned and the mesh element discretization density was increased around the excavation to employ a pseudo-radially gradational model mesh. As discussed in Chapter 3, the smallest mesh element measured no less than 3% of the tunnel radius (Walton and Diederichs 2015).

6.1.1.2.3 Model Geometry

For the purposes of this investigation, various tunnel shapes were assessed in order to gather an understand of their impact on the EDZ propagation within a brittle rockmass at depth. The first of the three tunnel geometries consist of a square tunnel measuring 3.2 m in width and height (see Figure 6-1). Square and circular tunnel profiles were chosen to serve as baseline geometries in assessing the effects of corner geometry on EDZ and baggage zone development.
These dimensions were selected in the interest of enabling correlation to the Mark II placement room tunnels discussed in Chapter 1.1.

To explore the effects of rounded corners on the predicted decrease in EDZ propagation, the corners of the square profile were rounded by a variety of lengths; 0.25 m increments were used to round the tunnel corners until a circular tunnel geometry was achieved. These various tunnel profiles can be seen below in Figure 6-2.

Figure 6-1: Dimensions of the trial square and circular tunnel excavations.

Figure 6-2: The various tunnel profiles used to explore the impacts of rounded corners. The length of rounding about each corner has been listed in center of each tunnel profile. Note that the tunnel profiles shown here are not to scale with respect to the dimensions shown in Figure 6-1.
6.2 Results and Discussion

Based on the orientation of the major and minor principal stresses, the extent (height) of the yielded elements was measured about the crown of the tunnel (refer to Figure 5-5 of Chapter 5) to quantify the EDZ development. Note that 0% and 100% corner rounding corresponds to the square and circular tunnel geometries, respectively.

6.2.1 Low Insitu Stress Regime

The impact of tunnel corner geometry on brittle damage and baggage zone development was first assessed at the lowest insitu stress regime (K of 1.5). The results presented in Figure 6-3 and Figure 6-4 display a correlation to previous results in Chapter 3 and Chapter 5 pertaining to the presence, or lack thereof, of a baggage zone in the crown of the tunnel at a K of 1.5.

Figure 6-3: Comparison of resulting EDZ development and lack of baggage zone when using Mohr-Coulomb versus Hoek-Brown failure criteria with a K of 1.5.
Figure 6-4: EDZ development (measured as per extent of yielded elements) about the tunnel geometries at a K of 1.5. Note that 0% and 100% rounding correspond to the square and circular geometries, respectively, while the intermediate data points correspond to the geometries shown in Figure 6-2.

The non-linearity of the results in Figure 6-4 (0 – 50% rounding) is a product of the fact that a complete baggage zone is not generated at low insitu stresses with minimal corner rounding; this result is even more apparent when modelling with Mohr-Coulomb failure criterion. This relationship can be observed in Chapter 5, as well as in Figure 6-3.

Upon increasing the degree of rounding about the tunnel corners, a complete baggage zone forms about the crown and floor of the excavation when using either failure criteria (see Figure 6-5). Note that although the formation of a baggage zone in brought on by increasing the degree of rounding about the tunnel corners, this rounding results in a reduction in the extent of yielded elements.
Mohr-Coulomb

Hoek-Brown

$K = 1.5$

0.75 m, 50% Rounding

1.0 m, 67% Rounding

1.25 m, 83% Rounding

100% Rounding

0.75 m, 50% Rounding

1.0 m, 67% Rounding

1.25 m, 83% Rounding

100% Rounding

Figure 6-5: Comparison of resulting EDZ development and complete baggage zone profiles when using Mohr-Coulomb versus Hoek-Brown failure criteria with a $K$ of 1.5.

Figure 6-6 on the following page presents the linear relationship between the degree of rounding and resulting extent of yielded elements about the crown of the tunnel previously shown in Figure 6-3 and Figure 6-5.
Figure 6-6: EDZ development (measured as per extent of yielded elements) about the tunnel geometries at a K of 1.5. Note that only tunnel geometries that presented a baggage zone about the tunnel crown have been graphed. The two equations depict the linear relationship between degree of rounding and resulting decrease in EDZ in relation to the failure criterion; $R^2$ values using these data points indicate good, linear fits.

Although the Hoek-Brown models are observed to present greater crown yield about square geometries at low insitu stress regimes (shown in Figure 6-3), the tunnel geometries modelled with Mohr-Coulomb failure criterion present a slightly greater extent in yielded elements about the tunnel crown. This can be attributed to the fact that the Mohr-Coulomb models present a sharper yield surface/baggage zone profile, as compared to the equivalent Hoek-Brown models.
6.2.2 Higher Insitu Stress Regimes

Each of the tunnel geometries were then assessed at higher insitu stress regimes to strengthen the previous conclusions. Similar to the results discussed in Chapter 3, the shape of the baggage zone was also analyzed for each of the geometries and stress regimes.

When modelled with a K of 1.75, each of the tunnel geometries, with the predicted exception of the circular tunnel, presented a baggage zone measuring roughly 20% of the height of the tunnel (see rectangular geometry in Figure 6-7). As shown in Figure 6-7, the percent was noted to decrease slightly upon achieving a near-circular tunnel profile.
Figure 6-7: Comparison of resulting EDZ development and baggage zone profiles when using Mohr-Coulomb versus Hoek-Brown failure criteria with a K of 1.75.
As shown by the linear fits in Figure 6-8, the extent of EDZ development about the tunnel crown, upon increasing the degree of rounding, is noted to gradually and continuously decrease by roughly the same percentage when modelling with either failure criteria.

![Figure 6-8: EDZ development (measured as per extent of yielded elements) about the tunnel geometries at a K of 1.75. The two equations depict the linear relationship between degree of rounding and resulting decrease in EDZ in relation to the failure criterion; $R^2$ values indicate good linear fits.](image)

Upon modelling the various tunnel geometries with a K of 2.0, the size of the baggage zone was noted to decrease due to the increase in yield magnitude and thickness of failure surface about the crown. The baggage zones measure 10% - 3% of the tunnel height, with the decrease in size attributed to the increased degree of rounding. This relationship is shown in Figure 6-9.
Figure 6-9: Comparison of resulting EDZ development and baggage zone profiles when using Mohr-Coulomb versus Hoek-Brown failure criteria with a K of 2.0.
Figure 6-10: EDZ development (measured as per extent of yielded elements) about the tunnel geometries at a K of 2.0. The two equations depict the linear relationship between degree of rounding and resulting decrease in EDZ in relation to the failure criterion; $R^2$ values indicate good linear fits.

Results presented in Figure 6-10 and later in Figure 6-12 agree with the conclusion initially drawn from Figure 6-8; the extent of EDZ development about the tunnel crown is noted to continuously decrease by roughly the same percentage when modelling with either failure criteria. Furthermore, this correlated decrease in yield due to increased corner rounding is independent of insitu stress regime (supported by the near identical linear fits in Figure 6-8, Figure 6-10 and Figure 6-12).

The following Figure 6-11 display the yield profiles generated about each of the tunnel geometries when modelled with a horizontal stress ratio of 2.5.
Figure 6-11: Comparison of resulting EDZ development and baggage zone profiles when using Mohr-Coulomb versus Hoek-Brown failure criteria with a \( K \) of 2.5
Figure 6-12: EDZ development (measured as per extent of yielded elements) about the tunnel geometries at a K of 2.5. The two equations depict the linear relationship between degree of rounding and resulting decrease in EDZ in relation to the failure criterion; $R^2$ values indicate good linear fits.

Regardless of the insitu stress regime, the tunnel geometries modelled with Mohr-Coulomb failure criterion present a slightly greater extent in EDZ development about the tunnel crown due to their sharper yield surface/baggage zone profile.

When modelling with a K of 2.5, a baggage zone only presented itself in the square tunnel geometry. As confirmed by Figure 6-11, the higher insitu stress regime results in continuous yield about the tunnel crown.

It is worth noting that results presented in Figure 6-11 highlight another key product of corner rounding. The degree of tensile failure in the tunnel walls is noted to decrease and
ultimately dissipate upon achieving a more-circular tunnel geometry. This is a result of the fact that circular tunnel profiles are known to promote more favorable stress redistribution about the excavation boundary.

6.3 Conclusions

At lower insitu stress regimes, rectangular/square geometries are again proven to be susceptible to the choice of Hoek-Brown versus equivalent Mohr-Coulomb failure criteria (see non-linearity in Figure 6-3 and differing yield profiles in Figure 6-4 and Figure 6-5). At high stress regimes, the choice of failure criteria is less deterministic in the resulting EDZ and baggage zone development.

The 2D FEM numerical modelling results presented in this chapter also prove that tunnel geometries modelled with Mohr-Coulomb failure criterion present a slightly greater extent in EDZ development about the tunnel crown due to their sharper yield surface/baggage zone profile; this is noted to occur regardless of insitu stress regime. This phenomenon can be attributed to the resulting shape of the two baggage zone profiles/surfaces, whereby which the Mohr-Coulomb models present a sharper profile, as compared to a more curved profile resulting from the use of Hoek-Brown failure criterion.

The rounding of the tunnel corners has been proven to result in a decrease in baggage zone size, while higher insitu stress regimes have been shown to result in continuous yield (no baggage zone) about the tunnel crown. In other words, higher stress states cause a reduction in the presence of a baggage zone, which is further compounded by the rounding of the tunnel corners. Additionally, rounded corners result in the dissipation of tensile failure within the excavation walls.
Chapter 7: The Impacts of 2-Dimensional versus 3-Dimensional FEM Numerical Modelling on Brittle Damage Development

7.1 Introduction

It has been proven that the degree of stress rotation can dramatically affect fracture propagation and subsequent damage development (Eberhardt 2001, Diederichs et al. 2004). It is evident that a 2D simulation cannot capture the complex stress rotations and variations that occur as the tunnel advances. To understand modelling discrepancies with respect to EDZ development in brittle rock, a comparison between the results obtained using RS² (2D) and RS³ (3D) has been conducted. A fixed point in the tunnel roof will provide the data for the following discussion.

7.1.1 Numerical Modelling: Input Parameters Model Set-up

7.1.1.1 Material and Strength Properties

Refer to Chapter 2.4 for Cobourg Limestone material properties. Figure 2-6 in Chapter 2.3 displays the DISL input parameters for the Hoek-Brown and equivalent Mohr-Coulomb failure criteria.

7.1.1.2 Stresses and Depth

As in the case of the 2D models presented in Chapter 3, each of the 3D models were assessed at a depth of 700 mbgs with identical stress conditions.

Table 7-1: Principal stresses used for each of the models discussed in this chapter.

<table>
<thead>
<tr>
<th>Stresses (MPa)</th>
<th>$K_h = 1.5$</th>
<th>$K_H = 2.0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Major Principal Stress ($\sigma_1$)</td>
<td>28.4</td>
<td>37.8</td>
</tr>
<tr>
<td>Minor Principal Stress ($\sigma_3$)</td>
<td>18.9</td>
<td>18.9</td>
</tr>
<tr>
<td>Out-of-Plane Stress ($\sigma_2$)</td>
<td>23.6</td>
<td>28.4</td>
</tr>
</tbody>
</table>
7.1.1.3 Numerical Modelling Software

For this chapter, two continuum modelling programs have been utilized. As members of RocScience, RS² version 9.0 and RS³ version 1.0 were used for 2D and 3D applications, respectively (see Chapter 2.2 for more information). See Appendix A for a complete list of models presented in Chapter 7.

7.1.1.4 Mesh Discretization, Boundary Conditions and Model Geometry

The model boundaries for the 2D and 3D numerical models presented in this chapter present a square model boundary that is 5x and 4x larger than the tunnel size, respectively. This was chosen to reduce boundary effects and to save computational time in the 3D models. The external boundaries were all pinned, which the exception of rollers used on the plane that contains the tunnel faces. The discretization density of the mesh elements around the excavation was again increased to employ a pseudo-radially gradational model mesh. As discussed in Chapter 4, the smallest mesh element measured no less than 3% of the tunnel radius (Walton and Diederichs 2015).

![Figure 7-1: RS³ – 3D FEM mesh with excavation stages indicated by blue markers.](image)
In RS\textsuperscript{3}, the true 3D excavation sequence was modelled with 1.0 m rounds. The 2D model staging (reduction in internal pressure to simulate tunnel advance) was done according to the longitudinal displacement profile (LDP) calibration approach of Vlachopoulos and Diederichs (2009).

7.2 Results and Discussion

For the first pass of this analysis, the rectangular tunnel with a K of 2.0 was used to calibrate the LDP. The evolution of boundary stress vs tunnel advance is shown in Figure 7-2.

Figure 7-2: Principal stress paths (in 2D and 3D) obtained from a fixed point in the rectangular tunnel roof. Note that the 2D results were calibrated with the rectangular, anisotropic tunnel model.
The results displayed in Figure 7-2 highlight the issues that arise when using a non-circular tunnel and an anisotropic stress state to calibrate the distances from the tunnel face. In contrast to the 3D results, the principal stresses of the 2D model are seen to experience the dramatic drop at a distance ahead the tunnel face.

As such, in correspondence with Vlachopoulos and Diederichs (2012), a circular tunnel with a radius of 1.6 m, along with isotropic stress conditions (37.8 MPa), was then used to calibrate the distance from the tunnel face at each stage of the RS$^2$ numerical model. These results have also been compared against those obtained from RS$^3$ (Figure 7-3) and show a better correlation of stress change for 2D and 3D models.

![Graph](image.png)

**Figure 7-3:** Principal stress paths (in 2D and 3D) obtained from a fixed point in the rectangular tunnel roof. Note that the 2D results were calibrated with the circular, isotropic tunnel model.
Note, however, the difference in the stress increase experienced in the 3D model as the face passes the reference point. This spike in stress can result in damage that would not appear in the 2D model.

This is evident in the final damage (yield and shear strain) plots from the 2D and 3D analysis (see Figure 7-4), as well as the results in Table 7-2. There are clear discrepancies in brittle damage development using 2D versus 3D models in high stress cases.

**Table 7-2: EDZ development when using RS² and RS³.**

<table>
<thead>
<tr>
<th>Geometry</th>
<th>Extent of EDZ (m)</th>
<th>Ratio (RS²:RS³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rectangle</td>
<td>2.1</td>
<td>0.89</td>
</tr>
<tr>
<td>Rounded Corners</td>
<td>1.4</td>
<td>1.0</td>
</tr>
<tr>
<td>Horseshoe</td>
<td>0.78</td>
<td>0.44</td>
</tr>
</tbody>
</table>

Figure 7-4: Comparison of EDZ and baggage obtained for 2D (left) and 3D (right) analyses of rectangular tunnel (using a K of 2.0 and Hoek-Brown DISL properties).
7.3 Conclusions

This study highlights numerous issues and challenges encountered when modelling brittle EDZ evolution with a continuum model. As previously discussed in several chapters, corners and flat boundaries create potentially undamaged baggage zones that have been observed in mining tunnels.

3D modelling shows, however, that in some cases, the stress changes around the advancing face creates a brittle damage zone where the undamaged baggage exists only in the 2D simulation (as highlighted in Figure 7-4). The spike in the 3D data in Figure 7-2 and Figure 7-3 is responsible for the continuous EDZ since the rockmass ahead of the tunnel undergoes yield. The complex stress rotations that occur as the tunnel advances cannot be captured in a 2D FEM model.

It is also important to be aware of the LDP limitations when calibrating a non-circular 2D model.
Chapter 8: A 3-Dimensional Assessment of the Impacts of Tunnel Shape, Orientation, Blast Round Length and Failure Criteria on Excavation Damage Zone Development in Brittle Rock

8.1 Introduction

8.1.1 Previous Modelling of Brittle Rock Failure in Tunnels

Numerous studies investigating the response of brittle rock to underground openings have been conducted. With the use of continuum modelling of a circular test tunnel, Martin et al. (1997) demonstrated how brittle rock failure generates v-shaped notches aligned with the orientation of the maximum compressive stress. Tensile regions were also observed at 90° to the v-shaped notches. As noted by Hajiabdolmajid et al. (2002), the slabbing process that generates these v-shaped notches, terminates upon achieving a more stable geometry. The authors attributed this phenomenon to the resulting increase in confinement and a subsequent decrease in plastic strain (which correlates to induced damage), thereby stopping the cohesive loss within the rockmass, ultimately leading to the termination of the v-shaped notch formation.

It is widely understood (Hoek and Brown 1980) that so-called “hard rocks” are normally 10 to 20 times weaker in tension than in compression. Thus, compressional loading, as in the case of underground tunnels and mine pillars, results in the generation of tensile cracking, under conditions where the tensile strength of the rockmass is reached, ultimately leading to spalling of brittle rock. Note that a critical crack density must be reached, for failure to take place. Diederichs et al. (2004) explain that until the damage initiation threshold is surpassed (CI threshold in Figure 1), these tensile fractures cannot propagate through the surrounding intact rockmass, thereby preventing stress induced damage. Diederichs (2000) listed a set of primary factors affecting crack interaction near underground excavations, which include: scale effects,
reduction in location confinement due to open cracks, crack surface interaction, pre-existing damage, damages due to stress rotations and heterogeneity and induced local tension.

Although modelled under 2D elastic conditions, Diederichs et al. (2004) proved that pre-existing cracks in the rockmass may be favorably oriented in a direction that lessens the need for new cracks to form, thereby making it easier to achieve the critical crack density necessary to initiate yielding. It was also concluded that stress rotations dramatically impact crack growth and coalescence. Diederichs et al. (2004) explain that favorably oriented stress rotation during tunnel advance can lead to crack extension, thereby promoting crack growth once the stresses rotate and return to their initial orientation. If similar stress rotations continue to take place, further crack propagation and eventual coalescence occurs.

Diederichs at al. (2004 and 2013) also correlated stress rotation to increased damage around underground excavations. As in the case of the Olmos Transandean tunnel in Peru, excessive rockmass yield occurred due to dramatic changes in the stress magnitudes and orientations along the length of the tunnel. Spalling and occasional bursting in the roof, with negligible face problems, took place while the major principal stress was aligned perpendicular to the tunnel axis. Geological structures (faulting) caused a near 90° stress rotation (major principal stress now aligned parallel with the tunnel axis), thereby resulting in face instability and crushing due to fracture propagation.

The results in this chapter will aim to support these conclusions by modelling a tunnel advance using various blast round lengths. Although the 3D continuum modelling software used for the purposes of this paper is not currently able to capture tensile crack development, near-face stress rotations and crack growth/coalescence, it is expected that these procedures are indeed occurring and impacting the final yield profile.
8.1.2 Numerical Modelling: Input Parameters and Model Set-up

8.1.2.1 Material and Strength Properties

Refer to Chapter 2.4 for Cobourg Limestone material properties. Figure 2-6 in Chapter 2.3 displays the DISL input parameters for the Hoek-Brown and equivalent Mohr-Coulomb failure criteria.

8.1.2.2 Stresses and Depth

Each of the various models were assessed at a depth of 700 mbgs. Major and minor principal stresses were calculated based on horizontal stress ratios ($K_{h,H}$) of 1.5 and 3.5. These values aim to depict a suitable range within the Canadian context (Al et al. 2011). Table 8-1 depicts the various stress regimes with respect to tunnel alignment.

Table 8-1: Principal stresses and tunnel alignment used for each of the models discussed in this chapter.

<table>
<thead>
<tr>
<th>Stress with Respect to Orientation</th>
<th>Stress (MPa)</th>
<th>Stress (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tunnel Aligned with a $K_{h}$ 1.5</td>
<td>Tunnel Aligned with a $K_{h}$ 3.5</td>
</tr>
<tr>
<td>Parallel to Tunnel Axis</td>
<td>28</td>
<td>66</td>
</tr>
<tr>
<td>Perpendicular to Tunnel Axis</td>
<td>66</td>
<td>28</td>
</tr>
<tr>
<td>Vertical ($\sigma_v$)</td>
<td>19</td>
<td>19</td>
</tr>
</tbody>
</table>

A schematic of the horizontal stresses has been presented in Figure 8-1 on the following page.
Figure 8-1: Illustration of the major and minor horizontal stress ratios ($K_{h,H}$) of 1.5 and 3.5. Note that modelling exercises included alternate trials with tunnel axis alignment collinear with either the major and minor horizontal stresses.

8.1.2.3 Numerical Modelling Software

$RS^3$ version 1.0 was used for each of the models discussed in this chapter (see Chapter 2.2 for more information).

8.1.2.4 Mesh Discretization and Boundary Conditions

For each of the numerical models presented in this chapter, a square model boundary, 4x larger than the tunnel size, was used to reduce boundary effects. The external boundaries were all pinned, while rollers were used on the external plane aligned with tunnel advance (XZ-plane).

To strike a balance between model efficiency and optimum mesh discretization, a mesh boundary was used. The size of this boundary measures half that of the total model boundary, thereby dividing the model into two evenly sized sections. A grading factor of 2 and an external grading factor of 0.5 was used for the entirety of the model. The mesh element density was then...
doubled within their interior portion of the model, generating mesh elements of 0.25 m in edge length along the excavation boundary. The discretization density of the mesh elements was increased around the excavation to employ a radially gradational model mesh, which aids to better delineate the extent of the EDZ around the tunnel perimeter (see Figure 8-2).

Figure 8-2: (A): RS³ – 3D FEM mesh. (B): 2D slice of the model face (XY-plane). (C): Highlight of mesh boundary and increased mesh element density around the excavation. Rectangular tunnel geometry is depicted.
8.1.2.5 Model Geometry and Blast Round Length

For the purposes of this investigation, various tunnel shapes were assessed to gather an understanding of their impact on the EDZ propagation within a brittle rockmass at depth. The first of the three tunnel geometries consisted of a rectangular tunnel geometry, measuring 5 m in width and 4 m in height.

To explore the effects of rounded corners on the predicted decrease in EDZ propagation, the corners of the aforementioned rectangular profile were rounded by 1.5 m. A horseshoe shaped tunnel, with a width and height if 5 m each, was also assessed. A schematic of these profiles is presented in Figure 8-3 below.

![Figure 8-3: Dimensions for the rectangular tunnel (left), rectangular tunnel with rounded corners (middle) and horseshoe tunnel profile (right).](image)

Finally, a circular tunnel geometry with a diameter of 5 m was modelled to simulate a tunnel advance using a tunnel boring machine (TMB). Model slices measuring 0.25 m in thickness were used to simulate a TBM advance. This slice thickness was chosen to correspond with the smallest mesh elements, previously described at 0.25 m in length along the excavation boundary.
The use of both rectangular/horseshoe tunnels, as well as a circular geometry, thereby enabled a comparative analysis between conventional drill-and-blast techniques, versus continuous TBM tunnel advance.

For ease of comprehension, Table 8-2 highlights each of the key components of the aforementioned 3D models.

### Table 8-2: Description of the various models assessed in this chapter.

<table>
<thead>
<tr>
<th>Model</th>
<th>Geometry</th>
<th>Blast Round Length</th>
<th>$K \parallel$ to Tunnel</th>
<th>$K \perp$ to Tunnel</th>
<th>Failure Criterion</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Circle</td>
<td>Continuous*</td>
<td>1.5 x $\sigma_v$</td>
<td>3.5 x $\sigma_v$</td>
<td>Hoek-Brown</td>
</tr>
<tr>
<td>2</td>
<td>Circle</td>
<td>Continuous*</td>
<td>3.5 x $\sigma_v$</td>
<td>1.5 x $\sigma_v$</td>
<td>Hoek-Brown</td>
</tr>
<tr>
<td>3</td>
<td>Rectangle</td>
<td>2.5 m</td>
<td>1.5 x $\sigma_v$</td>
<td>3.5 x $\sigma_v$</td>
<td>Hoek-Brown</td>
</tr>
<tr>
<td>4</td>
<td>Rectangle</td>
<td>2.5 m</td>
<td>3.5 x $\sigma_v$</td>
<td>1.5 x $\sigma_v$</td>
<td>Hoek-Brown</td>
</tr>
<tr>
<td>5</td>
<td>Rounded Corners</td>
<td>2.5 m</td>
<td>1.5 x $\sigma_v$</td>
<td>3.5 x $\sigma_v$</td>
<td>Hoek-Brown</td>
</tr>
<tr>
<td>6</td>
<td>Rounded Corners</td>
<td>2.5 m</td>
<td>3.5 x $\sigma_v$</td>
<td>1.5 x $\sigma_v$</td>
<td>Hoek-Brown</td>
</tr>
<tr>
<td>7</td>
<td>Horseshoe</td>
<td>2.5 m</td>
<td>1.5 x $\sigma_v$</td>
<td>3.5 x $\sigma_v$</td>
<td>Hoek-Brown</td>
</tr>
<tr>
<td>8</td>
<td>Horseshoe</td>
<td>2.5 m</td>
<td>3.5 x $\sigma_v$</td>
<td>1.5 x $\sigma_v$</td>
<td>Hoek-Brown</td>
</tr>
<tr>
<td>9</td>
<td>Horseshoe</td>
<td>1.5 m</td>
<td>1.5 x $\sigma_v$</td>
<td>3.5 x $\sigma_v$</td>
<td>Hoek-Brown</td>
</tr>
<tr>
<td>10</td>
<td>Horseshoe</td>
<td>1.5 m</td>
<td>3.5 x $\sigma_v$</td>
<td>1.5 x $\sigma_v$</td>
<td>Hoek-Brown</td>
</tr>
<tr>
<td>11</td>
<td>Horseshoe</td>
<td>3.5 m</td>
<td>1.5 x $\sigma_v$</td>
<td>3.5 x $\sigma_v$</td>
<td>Hoek-Brown</td>
</tr>
<tr>
<td>12</td>
<td>Horseshoe</td>
<td>3.5 m</td>
<td>3.5 x $\sigma_v$</td>
<td>1.5 x $\sigma_v$</td>
<td>Hoek-Brown</td>
</tr>
<tr>
<td>13</td>
<td>Horseshoe</td>
<td>3.5 m</td>
<td>1.5 x $\sigma_v$</td>
<td>3.5 x $\sigma_v$</td>
<td>Mohr-Coulomb</td>
</tr>
<tr>
<td>14</td>
<td>Horseshoe</td>
<td>3.5 m</td>
<td>3.5 x $\sigma_v$</td>
<td>1.5 x $\sigma_v$</td>
<td>Mohr-Coulomb</td>
</tr>
</tbody>
</table>

*Continuous excavation conducted using 0.25 m slices.

The length of the model in RS³ was set to 6 times the tunnel span/diameter to properly capture stress flow and yield ahead of and behind the tunnel face. Figure 8-4 on the following page depicts of the staging and model size.
Figure 8-4: RS³ – 3D FEM model with stages indicated by blue markers. The rectangular tunnel geometry is depicted, along with an inset of a partially excavated tunnel (contours of $\sigma_1$ shown).

Due to the complexities with modelling blast-induced damages, note that none of the modelling results in this paper include such influence.

8.2 Results and Discussion

8.2.1 EDZ₀ Development of Continuous Excavation (TBM) versus Drill-and-Blast

A circular tunnel geometry with a diameter of 5 m was modelled to simulate a tunnel advance using a TMB. These results have been compared against those obtained from modelling non-circular tunnel advances with 2.5 m blast rounds. Two different stress regimes have been utilized.

8.2.1.1 Yield Ahead of the Tunnel Face

RS¹ modelling results suggest that a continuously excavated tunnel generates less yielded rockmass ahead of an advancing tunnel face whose axis is aligned with the minor in-plane horizontal stress.
Although the circular tunnel presents slightly different dimensions in comparison to the other tunnel profiles, the circular pseudo-TBM model sees a 0.8 m and 0.4 m reduction in yielded rockmass, as compared to the horseshoe and rectangular shaped tunnels respectively. It should be noted that the continuously excavated circular tunnel and staged advance of the rectangular tunnel with rounded corners present nearly identical yield profiles ahead of the tunnel face. These results can be seen in Figure 8-5 below.

![Diagram showing EDZo development (m) about the tunnel face upon using different excavation methodology. (YZ-plane view is shown). Note the horizontal exaggeration.](image)

**Figure 8-5: EDZo development (m) about the tunnel face upon using different excavation methodology (YZ-plane view is shown). Note the horizontal exaggeration.**

Modelling results of tunnels aligned with the major in-plane horizontal stress present less definitive conclusions. This would suggest that tunnels aligned in such fashion are less dependent of excavation methodology compared to 2D shape details (tunnel profile).
8.2.1.2 Circumferential/Perimeter Tunnel Yield at the Tunnel Face

As predicted, 3D modelling results indicated that the circular pseudo-TBM model generates far less yielded rockmass about the tunnel face perimeter. The continuously excavated tunnel results in a 0.7 m, 1.3 m and 0.5 m decrease in yield about the tunnel crown, floor and walls, respectively, when aligned with the minor in-plane horizontal stress. Upon realignment of the tunnel axes with the major in-plane horizontal stress, the crown, floor and walls present 0.3 m, 0.7 m and 0.2 – 0.6 m less yielded rockmass in the continuously excavated tunnel model. Due to the curved geometry of the circular tunnel floor, the localized decrease in yield due to favorable stress redistribution was expected. However, these modelling results should be attributed to 2D shape effects (i.e. tunnel profile), rather than a product of excavation methodology.

The most noteworthy result of employing different excavation methodologies is the yield profile generated along the length of the tunnel walls when aligned with the minor in-plane horizontal stress. As shown in Figure 8-6, the continuously excavated circular tunnel presents uniform yield extending radially from the walls (Models 1 and 2). Conversely, the 2.5 m blast rounds result in a dramatically different top-down profile. The walls experience increased yield at each 2.5 m increment (i.e. where the face is located after each blast round), while the intermediate wall sections present less radially yielded rockmass (Model 7), compared to the continuously excavated tunnel (Model 1).
Figure 8-6: EDZ₀ development about the tunnel walls and face for the various models. Top-down view shown (along with tunnel advance direction); yellow indicates yielded elements. Refer to Figure 8-1 for colour coding. Left = tunnels aligned with Kₗ of 1.5. Right = tunnels aligned with Kₗ of 3.5. BR = blast round.

Due to predictable stress redistribution around circular openings (Hoek and Brown 1980), excavation methodology is less critical in terms of resulting wall yield when the tunnel axis is aligned with the major in-plane horizontal stress (excessive yield in Models 2 and 8 in Figure 8-6).

Figure 8-7 on the following page further supports the aforementioned conclusions, as well as the illustrated data presented in Figure 8-6 above.
When aligning the tunnel axes with the minor in-plane horizontal stress, the circular tunnel profile acts to reduce the EDZ_o development about the crown and floor at the tunnel face by 0.7 m and 1.1 m, respectively. It should be noted that the larger, more pronounced, difference in floor yield is likely product of 2D shape effects in addition to excavation methodology; the circular profile acts to better redistribute stresses in comparisons to the 90° bottom corners of the horseshoe tunnel. The circular, continuously excavated tunnel also presents roughly 0.2 – 0.4 m less yield about the tunnel walls. Again, the overall tunnel profile discrepancy between the two models is likely contributing to the variation in wall yield.
Upon re-aligning the tunnels with the major in-plane horizontal stress, it is observed that the two excavation methodologies present more similar results with respect to circumferential/perimeter EDZ\(_o\) development at the tunnel face. Although the continuously excavated tunnel still results in a decrease in yield rockmass about the face perimeter, the decrease is less significant upon tunnel alignment with the major in-plane horizontal stress. This phenomenon can be attributed to the fact that the 2-dimensional rounding of the tunnel profile is not aligned with the orientation of maximum compressive stress. This thereby results in a significantly lower impact on stress redistribution perpendicular to the tunnel axis.

8.2.1.3 Circumferential/Perimeter Tunnel Yield at Three Tunnel Spans Back from the Face

Literature suggests that the maximum radius of yielded rockmass occurs roughly three tunnel spans back from the face (Hoek and Brown 1980). To therefore conduct a complete and in-depth analysis of the impact of 2D shape effects on EDZ\(_o\) development around an advancing tunnel, the yield about the tunnel perimeter was also assessed at roughly three spans back from the face.

Since Figure 8-6 highlights the fact that the tunnel walls experience increased yield at each 2.5 m increment (i.e. where the face is located after each blast round), while the intermediate wall sections present less radially yielded rockmass (see Model 7 in Figure 8-6), two comparative studies were conducted. As presented in the following sections, the perimeter yield was assessed at roughly three tunnel spans back from the advancing face at both the front of a blast round (i.e. 6 blast round or 15 m back from the face), as well as the mid-section of a blast round (i.e. 5.5 blast rounds or 13.75 m back from the face). These locations are highlighted by the red and green lines in Figure 8-6, respectively.
8.2.1.3.1 Circumferential/Perimeter Tunnel Yield at the Front of a Blast Round

Recall that this location is highlighted by the green lines in Figure 8-6 above.

3D modelling results support the aforementioned research; the radius of yielded rockmass about the tunnel circumference/perimeter increases upon assessing a tunnel region greater than three spans back from the face.

When comparing the yield about the tunnel crown and floor at the face to a distance of six blast rounds back from the face, the increase in EDZ₀ development appears to vary depending on excavation methodology. At six blast rounds back from the tunnel face, the yield about the crown and floor of the continuously excavated, pseudo-TBM model increases by 1.2 – 1.4 m when aligned with the minor in-plane horizontal stress, and 0.9 – 1.2 m when aligned with the major in-plane horizontal stress. Although the yield about the crown and floor of the drill-and-blast horseshoe tunnel does indeed increase upon assessing a slice six blast rounds back from the face, the increase in the extent in EDZ₀ development appears to be far more dependent upon the in-plane horizontal stress. When aligned with the minor in-plane horizontal stress, yield in the crown and floor increases by 0.7 – 1.0 m, while an increase of 1.6 – 1.7 m is noted to occur in the crown and floor of the tunnel aligned with the major in-plane horizontal stress. See Figure 8-7 and Figure 8-8 for a graphical comparison of the aforementioned results.

A comparative analysis of the EDZ₀ development about the tunnel walls was also assessed at both locations. Tunnels aligned with the minor in-plane horizontal stress present the same wall yield at the face as compared to six blast rounds back from the face. Conversely, tunnels aligned with the major in-plane horizontal stress see a dramatic increase in the wall yield at six blast rounds back from the face. The continuously excavated, pseudo-TBM model presents a 0.9 m increase in wall yield at six blast rounds back from the face, while the wall yield about the drill-and-blast horseshoe tunnel increases by 1.6 – 1.8 m at six blast rounds back from the face. These results are made evident by assessing the yield profiles in Figure 8-7 and Figure 8-8.
8.2.1.3.2 Circumferential/Perimeter Tunnel Yield at a Blast Round Mid-Section

Recall that this location is highlighted by the red lines in Figure 8-6 above.

Quantitative comparisons of the results presented in Figure 8-7 and Figure 8-9 indicate that the yielded rockmass about the tunnel crown and floor increases by $0.9 - 1.9$ m when assessing a slice at 5.5 blast rounds (mid-section of a blast round) back from the tunnel face for both excavation methodologies. Similar to the results presented in the previous section, the drill-and-blast horseshoe tunnel presents a far larger increase in the extent in EDZ$_o$, while the magnitude is again dependent on the tunnel alignment; drill-and-blast tunnels see a larger
increase in yield about the crown and floor at 5.5 blast rounds back from the face, as compared to the face, when aligned with the major in-plane horizontal stress.

V-shaped notches are still noted to form in the direction of maximum compressive stress, regardless of tunnel geometry, although slightly lessened by the complete rounding of the circular tunnel versus the horseshoe profile.

Figure 8-9: (A): $EDZ_0$ development (m) about the tunnel face circumference/perimeter for the various models. (B): $EDZ_0$ development (m) about the tunnel perimeter at six blast rounds back from the tunnel face. (C): $EDZ_0$ development (m) about the tunnel perimeter at 5.5 blast rounds back from the tunnel face.

When aligned with the major in-plane horizontal stress, the continuously excavated, pseudo-TBM model presents a 0.9 m increase in yield about the walls, while the drill-and-blast tunnel sees a 1.5 – 2.0 m increase in wall yield at 5.5 blast rounds back from the tunnel face.

Figure 8-7 and Figure 8-9 graphically display these significant changes.

Since it is previously understood that intermediate wall sections present less radially yielded rockmass (Model 7 in Figure 8-6) when aligned with the minor in-plane horizontal stress,
the following results thereby correlate to this aforementioned conclusion. For continuously excavated tunnels aligned with the minor in-plane horizontal stress (solid lines), the wall yield remains the same along the entire length of the tunnel. However, the drill-and-blast horseshoe tunnel presents 0.4 - 0.5 m less wall yield at 5.5 blast rounds spans back from the advancing tunnel face.

8.2.1.4 Conclusions

Results presented in Chapter 8.2.1 suggest that excavation methodology dramatically impacts the profile of yielded rockmass along the tunnel wall, especially when assessing tunnels aligned with the minor in-plane horizontal stress.

It can also be concluded that tunnel alignment per major and minor-in plane horizontal stress is of greater impact on increased EDZo development when assessing drill-and-blast tunnel excavation as compared to continuously excavated tunnels.

The aforementioned 3D modelling results generally support the research presented by Hoek and Brown (1980); the radius of yielded rockmass about the tunnel circumference/perimeter increases and reaches it maximum upon at roughly three spans back from the tunnel face.

8.2.2 The Impacts of 2D Shape Effects on EDZo Development

To explore the effects of 2D tunnel profile on EDZo propagation, three different tunnel geometries were modelled using 2.5 m blast rounds under two different stress regimes.

8.2.2.1 Yield Ahead of the Tunnel Face

The rounding of the tunnel corners also acts to reduce the yielded rockmass ahead of the advancing tunnel face by 0.2 – 0.8 m for tunnels aligned with the minor in-plane horizontal stress.

When assessing tunnels whose axes are aligned with the major in-plane horizontal stress, the rounded corner geometry conversely results in a 0.1 - 0.7 m increase in EDZo development ahead of the face, primarily concentrated along the rounded edges. This result suggests that the
2D rounding in relation to 3D stress flow is the driving factor leading to the increased tunnel face yield in front of the corners (see isolated regions in Model 6 of Figure 8-10).

Figure 8-10 also highlights a significant product of wall yield resulting from 2D tunnel profile. When comparing Model 3 (rectangle) and 5 (rounded corners), it is apparent that when the tunnel axis is aligned with the minor in-plane horizontal stress, portions of the wall between the 2.5 m incremental blast rounds of the rounded tunnel do not yield (blue portions). As anticipated, the entire wall of both tunnel geometries undergoes failure upon tunnel realignment with the major in-plane horizontal stress (Models 4 and 6 in Figure 8-10).

![Figure 8-10: EDZ\textsubscript{0} development about the advancing tunnel for the various models. Side view shown (along with tunnel advance direction); yellow indicates yielded elements. Left = tunnels aligned with $K_h$ of 1.5. Right = aligned with $K_H$ of 3.5.](image)
8.2.2.2 Perimeter Tunnel Yield at the Face

Results of the 3D continuum analysis suggest that tunnel profiles that approach a more circular geometry present less EDZo development about the tunnel perimeter. As predicted, the rectangular tunnel profile generates the greatest yield about the tunnel crown, floor and walls. Corresponding with the research conducted by various aforementioned authors (Martin et al. 1997, Hajiabdolmajid et al 2002, Diederichs et al. 2013), brittle rockmass failure results in the formation of v-shaped notches in both the floor and the crown of the tunnels. As anticipated, an increase in the size of the v-shaped notch occurs when the major in-plane horizontal stress runs perpendicular to the tunnel alignment.

The 1.5 m rounding of the corners leads to a dramatic reduction in the EDZo development about the tunnel perimeter when aligned with the minor in-plane horizontal stress. A 1.0 m reduction in yield about the crown and floor (reduction in size of v-shaped notch), as well as a 0.6 m reduction in the tunnel walls occurs.

When aligning the tunnel axes with the major in-plane horizontal stress, the rounded corners are observed to have a far lesser effect on minimizing EDZo development. This result can be attributed to the fact that the 2-dimensionally rounded corners are not aligned with the orientation of maximum compressive stress. This thereby results in a significantly lower impact on stress redistribution perpendicular to the tunnel axis.

The extent of damage in the crown remains the same, while the floor and walls see a mere 0.3 m and 0.2 m reduction respectively in yielded rockmass upon rounding of the corners. These results are presented in Figure 8-11.
8.2.2.3 Perimeter Tunnel Yield at Three Tunnel Spans Back from the Tunnel Face

Since research presented in the previous section highlights the fact that the tunnel walls experience increased yield at the face of each blast round, while the intermediate wall sections present less radially yielded rockmass, a similar analysis was conducted in order to investigation this correlation to 2D shape effects.

Once again, the perimeter yield was assessed at roughly three tunnel spans back from the advancing face at both the front of a blast round (i.e. six blast round or 15 m back from the face), as well as the mid-section of a blast round (i.e. 5.5 blast rounds or 13.75 m back from the face). These locations are highlighted by the red and green lines in Figure 8-10, respectively.
8.2.2.3.1 Perimeter Tunnel Yield at the Front of a Blast Round

The yield about the tunnel perimeter was also assessed at roughly three spans or six blast rounds (i.e. 15 m) back from the face. This location is highlighted by the green lines in Figure 8-10 above.

In accordance with Hoek and Brown (1980), the overall yield profile at roughly three spans back from the tunnel face is significantly larger than at the tunnel face. A quantitative comparison of the results presented in Figure 8-11 and Figure 8-12 reveal that the crown and floor of the rectangular and rounded tunnel experience a 0.7 – 2.1 m and 0.9 – 1.7 m increase, respective of shape, when assessing a slice a roughly three spans back from the face. It is noted that a slightly larger increase in yielded rockmass is observed in tunnels whose axes are aligned with the major in-plane horizontal stress.

Figure 8-12: (A): EDZo development (m) about the tunnel face perimeter for the various models. (B): EDZo development (m) about the tunnel perimeter at six blast rounds back from the tunnel face.
The increasing yield about the tunnel walls presents compelling results in terms of both tunnel alignment with respect to major and minor in-plane horizontal stress, as well as 2D shape effects. When the tunnels are aligned with the major in-plane horizontal stress, EDZ development about the wall of the rectangular and rounded tunnel increases by 1.6 – 1.8 m and 1.1 – 1.2 m, respectively, when assessing the front of a blast round roughly three spans back from the face. The lessened increase in yielded rockmass in the rectangular tunnel with the rounded corners is a direct product of the 2D rounding, resulting in more favourable stress flow, and a decrease in the magnitude of stress concentration about the tunnel perimeter.

Wall yield at the front of a blast round three spans back from the face is equal to that measured at the tunnel face when modelled parallel to the minor in-plane horizontal stress. This also correlates to the aforementioned results pertaining the circular pseudo-TBM tunnel, as well as the horseshoe-shaped drill-and-blast tunnel.

### 8.2.2.3.2 Perimeter Tunnel Yield at a Blast Round Mid-Section

The yield about the tunnel perimeter was lastly assessed at roughly three spans or 5.5 blast rounds (i.e. 13.75 m) back from the face. This location is highlighted by the red lines in Figure 8-10 above.

Modelling results prove that the radius of yielded rockmass about the tunnel perimeter does indeed increase dramatically upon assessing a tunnel region approximately three spans back from the face. Similar to the results obtained from the previous section (front of a blast round), the yielded rockmass about the tunnel crown and floor of the rectangular and rounded tunnel experiences a 0.9 – 2.1 m and 1.2 – 1.6 m increase, respective of shape. Furthermore, a slightly larger increase in yielded rockmass is again observed in tunnels whose axes are aligned with the major in-plane horizontal stress (especially the rectangular tunnel).
Figure 8-13: (A): EDZ\textsubscript{o} development (m) about the tunnel face perimeter for the various models. (B): EDZ\textsubscript{o} development (m) about the tunnel perimeter at six blast rounds back from the tunnel face. (C): EDZ\textsubscript{o} development (m) about the tunnel perimeter at 5.5 blast rounds back from the tunnel face.

The quantitative results of the EDZ\textsubscript{o} development about the tunnel walls strongly correlates to aforementioned results. When the tunnels are aligned with the major in-plane horizontal stress, EDZ\textsubscript{o} development about the wall of the rectangular and rounded tunnel increases by 2.0 – 2.1 m and 1.0 – 1.1 m, respectively, when assessing the front of a blast round roughly three spans back from the face. This finding further supports the initial conclusion that the 2D rounding of the rectangular tunnel corners effectively reduces the EDZ\textsubscript{o} propagation about the tunnel perimeter at both the face and along its entire length.

Based on the previous understanding that portions of the wall between the incremental blast rounds of the rounded tunnel do not yield when the tunnel axis is aligned with the minor in-plane horizontal stress (Figure 8-10 and later Figure 8-16), it is agreeable that the extent of wall yield about the mid-section of the blast round is slightly smaller than at the face of a blast round.
As such, the rectangular and rounded tunnel geometries experience a 0.1 – 0.3 m decrease in EDZ₀ development when assessing the mid-section.

8.2.2.4 Conclusions

The results discussed throughout Chapter 8.2.2 globally indicate that 2D tunnel shape effects, such as rounding of rectangular corners, have a more pronounced impact in the reduction of yielded rockmass, when the tunnel is aligned with the minor in-plane horizontal stress (i.e. perpendicular to the direction of maximum compressive stress). Under this alignment, the rounding of the tunnel corners also acts to reduce the yielded rockmass ahead of the advancing tunnel face.

It was also determined that the wall yield profile along the length of the tunnel is highly dependent on the tunnel orientation with respect to in-plane horizontal stress ratio. Although a slight reduction in wall yield occurs when rounding the tunnel corners, the resulting wall yield is relatively irrespective of tunnel geometry.

This study further supports the initial conclusion that the 2D rounding of the rectangular tunnel corners effectively reduces the EDZ₀ propagation about the tunnel perimeter at both the face and along its entire length.

Finally, v-shaped notches are again noted to form in the direction of maximum compressive stress, regardless of tunnel geometry, although lessened by the rounding.

8.2.3 The Impact of Blast Round Length on EDZ₀ Development

Three different blast round lengths measuring 1.5 m, 2.5 m and 3.5 m were used to determine their relationship in terms of yield propagation ahead of an advancing tunnel.
8.2.3.1 Yield Ahead of the Tunnel Face

Modelling indicates that the blast round length does not play a significant role in the development of yielded rockmass ahead of an advancing tunnel face, regardless of the in-plane horizontal stresses.

Alignment of the tunnel axes with major and minor in-plane horizontal stresses is again the dominant factor in EDZ development ahead of the tunnel face. Each of the three blast round lengths generates 2.8 – 2.9 m of yielded rock ahead of the center of an advancing tunnel aligned with the minor in-plane horizontal stress (see solid lines in Figure 8-14). However, 0.05 m of yielded rockmass has been observed ahead of the central axis of each of the three tunnels when aligned with the major in-plane horizontal stress (dashed lines in Figure 8-14). This is a near 100% reduction in yield ahead of the tunnel face upon favorable change in tunnel alignment. Additionally, the crown and floor experience a 1.1 – 1.3 m and 1.5 – 2.0 m reduction in yield ahead of the advancing face respectively.

It is worth noting that upon alignment of the tunnel with the major in-plane horizontal stress, the rockmass near the crown, floor and tunnel walls experiences an increase in yield ahead of the tunnel face, compared to the center of the face. The corner bulges of the dashed lines in Figure 8-14 highlight these results.
Figure 8-14: EDZ_o development (m) about the tunnel face upon using different blast rounds (side profile (YZ-plane) view is shown). Note the horizontal exaggeration.

8.2.3.2 Perimeter Tunnel Yield at the Face

3D continuum modelling results also suggest that the blast round length plays a negligible role in the reduction of EDZ_o development about the tunnel face perimeter. Figure 8-15 highlights these findings (grouping of solid and dashed lines).
Figure 8-15: EDZo development (m) about the tunnel face perimeter upon using different blast round lengths.

Of importance however, is the alignment of the tunnel axis with the in-plane horizontal stresses. For horseshoe shaped profiles, tunnels aligned with the minor in-plane horizontal stress experience a 0.9 – 1.9 m reduction in yield about the crown and a 1.2 – 1.4 m reduction in yield about the floor, compared to equivalent tunnels aligned with the major in-plane horizontal stress. The yielded rockmass about the walls remains relatively constant despite the change of in-plane horizontal stress. Furthermore, the rounding of the crown lessens the size of the v-shaped notches, compared to the floor, due to favorable stress flow.
8.2.3.3 Perimeter Tunnel Yield at Three Tunnel Spans Back from the Face

In order to maintain continuity with the previous sections in this chapter and to develop a full understanding of the impact of blast round length on EDZo along the entire length of an advancing tunnel, the perimeter yield was also assessed at roughly three spans back from the perimeter face. Note that since this portion of the investigation models various blast round lengths, the distance back from the tunnel face varies slightly with each model.

The green lines in Figure 8-16 on the following page depict the locations of the front of the blast rounds assessed at roughly three tunnel spans back from the face. This slice is located at 15 m, 15 m and 14 back from the face for 1.5 m BR, 2.5 m BR and 3.5 m BR models, respectively. The red lines highlight the location of the mid-section of each blast round assessed at roughly three tunnel spans back from the face. These slices are located at 14.25 m, 13.75 m and 15.75 m back from the face for the 1.5 m BR, 2.5 m BR and 3.5 m BR models, respectively.
Figure 8-16: EDZ\textsubscript{o} development about the advancing tunnel for the various models. Side view shown (along with tunnel advance direction); yellow indicates yielded elements. Top = 1.5 m blast round (RB). Middle = 2.5 m BR. Bottom = 3.5 m BR. Note that all tunnels shown are aligned with a $K_h$ of 1.5.
8.2.3.3.1 Perimeter Tunnel Yield at the Front of a Blast Round

Upon assessment of the perimeter yield results at the front of a blast round located roughly three spans back from the tunnel face, it is again apparent that the blast round length plays an insignificant role in the resulting EDZ_0 development about the tunnel perimeter. This is made evident by the grouping of the solid and dashed lines in Figure 8-17 below, whereby a mere 0.1 – 0.4 m deviance in yield rockmass is noted to occur. The size and location of the v-shaped notes are also independent of blast round length.

Figure 8-17: (A): EDZ_0 development (m) about the tunnel face perimeter upon using different blast round lengths. (B): EDZ_0 development (m) about the tunnel perimeter at six blast rounds back from the tunnel face.

As is to be expected (Hoek and Brown 1980), the overall yield profile at roughly three spans back from the tunnel face is significantly larger than at the face itself; the crown and floor
see a 0.7 – 1.8 m and 0.9 – 1.7 m increase when assessing a slice at roughly three spans back from the face. Similar to the rectangular and rounded tunnel geometries, a slightly larger increase in yielded rockmass about the crown and floor is observed in tunnels whose axes are aligned with the major in-plane horizontal stress. Yield about the tunnel walls is noted to increase by up to 2.3 m when assessing the slice at roughly three spans back from the tunnel face.

In agreement with the previous section, the alignment of the tunnel axis with the in-plane horizontal stresses is again the primary factor in shape and extent of EDZo development about the tunnel perimeter. A tunnel aligned with the minor in-plane horizontal stress experiences up to a 0.6 m reduction in yield about the crown and a 0.6 – 1.4 m reduction in yield about the floor, compared to equivalent tunnels aligned with the major in-plane horizontal stress. In accordance with known stress flow with respect to major and minor stress magnitudes, the tunnels aligned with the major in-plane horizontal stress are noted to have 1.4 – 2.4 m less of yielded rockmass about the tunnel walls.

V-shaped notches are noted to form in the direction of maximum compressive stress, regardless of blast round length.

**8.2.3.3.2 Perimeter Tunnel Yield at a Blast Round Mid-Section**

A slice located 14.25 m, 13.75 m and 15.75 m back from the tunnel face will serve as the reference point for the 1.5 m, 2.5 m and 3.5 m blast round models, respectively. Each of these three locations are approximately three tunnel spans (5.5 blast rounds) back from the face.

Upon quantitively comparing the results presented in Figure 8-15 and Figure 8-18, it is apparent that the extent of increased yielded rockmass about the tunnel perimeter is again predominantly related to the alignment of the tunnel axis with the in-plane horizontal stresses. This is again highlighted by the grouping of solid and dashed lines.
Figure 8-18: (A): EDZ_o development (m) about the tunnel face perimeter upon using different blast round lengths. (B): EDZ_o development (m) about the tunnel perimeter at six blast rounds back from the tunnel face. (C): EDZ_o development (m) about the tunnel perimeter at 5.5 blast rounds back from the tunnel face.

The results presented above in Figure 8-18 support previously published research (Hoek and Brown 1980) and results presented in this chapter which states that the maximum radius of yielded rockmass occurs roughly three tunnel spans back from the face. The crown and floor experience a 0.6 – 1.8 m and 1.0 – 1.9 m increase when assessing a slice at roughly three spans back from the face. Note that this is nearly identical to the aforementioned results taken from the front of a blast round roughly three spans back from the face.

Of importance is the yield development about the tunnel walls. While the front of the blast round located three spans back from the face experiences up to a 2.3 m increased in yield about the walls, the results of the blast round mid-section roughly three spans back form the tunnel face varies slightly.
For tunnels aligned with the major in-plane horizontal stress, EDZₒ development about the walls is 1.5 m – 2.2 m larger than at the tunnel face, and nearly identical to the walls of the slice assessed at the front of a blast round three spans back from the face.

However, the blast round length appears to be correlated to the extent of yielded rockmass about the wall for tunnels aligned with the minor in-plane horizontal stress (note the larger spacing in the solid lines versus dashed lines in Figure 8-18). This is in accordance with the previous understanding that when the tunnel axis is aligned with the minor in-plane horizontal stress, portions of the wall between the incremental blast rounds of the rounded tunnel do not yield (Figure 7-10 and Figure 7-16). As such, the 1.5 m and 2.5 m blast round models experience 0.1 m – 0.5 m less yield about the tunnel wall as compared to the tunnel face and the front of a blast round three spans back from the face.

As shown in Figure 8-18, v-shaped notches are again noted to form in the direction of maximum compressive stress, regardless of blast round length.

8.2.3.4 Conclusion

The 3D FEM modelling results presented in Chapter 8.2.3 globally indicate that blast round length does not play a significant role in the development of yielded rockmass ahead of an advancing tunnel face, and is also unrelated to the in-plane horizontal stresses. Similar to the results discussed in Chapter 7.2.2, alignment of the tunnel axes per major and minor in-plane horizontal stresses is again the dominant factor in EDZₒ development ahead of the tunnel face.

Results also suggest that EDZₒ development about the tunnel face perimeter is also unassociated to blast round length.

Although V-shaped notches are still noted to form in the direction of maximum compressive stress (regardless of blast round length), the rounding of the crown compared to the
rectangular geometries assessed in Chapter 8.2.3 lessens the size of the v-shaped notches due to favorable stress flow.

The blast round length does, however, appear to be a slightly deterministic factor in the extent of yielded rockmass about the wall for tunnels aligned with the minor in-plane horizontal stress. Note that this conclusion correlates to the previous understanding made in Chapter 7.2.2 in regard to the fact that portions of the wall between the incremental blast rounds of the rounded tunnel do not yield.

8.2.4 Effects of Generalized Hoek-Brown DISL versus Equivalent Mohr-Coulomb DISL on EDZ Development

Previous modelling investigations conducted by the authors (Cain and Diederichs 2016) indicate that for staged excavations simulated in RS² (two-dimensional), lower stress regimes (K of 1.5) are far more sensitive to the use of different failure criteria. Tunnels modelled with generalized Hoek-Brown failure criterion present approximately double to triple the amount of yield around the tunnel corners, compared to tunnels assessed with equivalent Mohr-Coulomb parameters. With an increasing stress regime (K of 2.0 and greater), nearly identical results were achieved when modelling with either failure criteria. RocScience’s RS³ was used to determine if the aforementioned conclusions hold true when modelled using 3D software. The horseshoe tunnel profile with 3.5 m blast rounds was modelled at both stress regimes.

8.2.4.1 Yield Ahead of the Tunnel Face

The yield about an advancing tunnel face was also analyzed. 3D modelling results using the minor horizontal stress ratio (K of 1.5) indicate that the Hoek-Brown models generate an increase of approximately 0.8 m of yielded rockmass ahead of an advancing tunnel face. Conversely, yield about the tunnel face decreased by 0.3 m when using Hoek-Brown properties at an increased in-plane horizontal stress.
When quantifying the extent of yielded rockmass ahead of an advancing 3D tunnel face, the Hoek-Brown failure criterion produces larger values when the tunnel is aligned with the minor in-plane horizontal stress, while 3D models using Mohr-Coulomb failure criterion conversely produce larger values when the tunnel is aligned with the major in-plane horizontal stress. These results are depicted in Figure 8-19.

![Diagram showing EDZ development](image)

**Figure 8-19:** EDZ\(_0\) development (m) about the tunnel face upon using different failure criteria (side profile (YZ-plane) view is shown). Note the horizontal exaggeration.
8.2.4.2 Perimeter Tunnel Yield at the Face

With the tunnel aligned with the minor in-plane horizontal stress, the Hoek-Brown model presents a yield profile that is 0.8 – 0.9 m and 0.1 - 0.2 m the larger about the crown and floor, and side walls at the face, respectively, as compared to the Mohr-Coulomb model. This relationship is shown by the solid lines in Figure 8-20.

![Figure 8-20](Image)

**Figure 8-20: EDZₗ₀ development (m) about the tunnel face perimeter caused by using different failure criteria.**

A tunnel aligned with an increased in-plane horizontal stress presents similar results. The Hoek-Brown model generates a face perimeter yield profile 0.1 – 0.2 m larger about the crown and floor, and 0.2 m larger about side walls, as compared to the Mohr-Coulomb model (dashed lines in Figure 8-20).
It therefore can be concluded that when assessing the perimeter yield profile at the face (as in Figure 8-20), tunnels aligned with the minor in-plane horizontal stress are far more sensitive to the use of specific failure criterion.

Overall, these results also suggest that the crown, floor and tunnel face are more sensitive to the failure criterion, compared to the tunnel walls, when quantifying the extent of yielded rockmass and EDZ development.

8.2.4.3 Perimeter Tunnel Yield at Three Tunnel Spans Back from the Face

Similar to each of the previous investigations, the perimeter yield about the horseshoe tunnel was assessed at roughly three tunnel spans back to the face. Since the models (models 11 through 14) assessed in this portion of the chapter utilized 3.5 m blast rounds, the perimeter yield at the front of a blast round roughly three span back is located at 14 m (four blast rounds) back from the tunnel face. The slices used to assess the perimeter yield at the mid-section of a blast round is located at 15.75 m (4.5 blast rounds) back from the tunnel face. Recall that these locations are highlighted by the green and red lines in Figure 8-6.

8.2.4.3.1 Perimeter Tunnel Yield at the Front of a Blast Round

A quantitative comparison of the results presented in Figure 8-20 and Figure 8-21 correlate to the initial research presented by Hoek and Brown (1980), as well as aforementioned results presented in this chapter. The crown and floor of the tunnel experience a 0.7 – 1.8 m and 1.2 – 1.6 m increase, respectively, when assessing a slice at the front of a blast round located roughly three spans back from the face (see Figure 8-21).

As is the case with the other results discussed in this chapter, the extent of increased yielded rockmass about the tunnel perimeter is again predominantly related to the alignment of the tunnel axis with the in-plane horizontal stresses. This is again highlighted by the grouping of solid and dashed lines in Figure 8-21. In correlation with the rectangular geometries, a slightly
larger increase of approximately 0.6 – 1.0 m in yielded rockmass about the crown and floor is observed in tunnels whose axes are aligned with the major in-plane horizontal stress.

Modelling results pertaining to wall yield also hold true to this conclusion; for tunnels aligned with the major in-plane horizontal stress, the wall yield increases by 1.7 – 1.9 m when comparing the front of a blast round at roughly three spans back form the face to the tunnel face itself (applicable to both failure criteria). The extend of yielded rockmass about the wall is noted to increase by a mere 0.1 – 0.3 m when assessing the tunnels whose axes are aligned with the minor in-plane horizontal stress.

![Figure 8-21](image)

**Figure 8-21:** (A): EDZ\textsubscript{o} development (m) about the tunnel face perimeter caused by the use of different failure criteria. (B): EDZ\textsubscript{o} development (m) about the tunnel perimeter at four blast rounds back from the tunnel face.
Although the results support the research presented by Hoek and Brown (1980) in terms of expected increase in yield at roughly three spans back from the face, it should be noted that the use of different failure criteria does impact the magnitude and extent of EDZ development.

Figure 8-21 highlights the fact that the use of the Hoek-Brown failure criterion results in a perimeter yield profile that is larger than that generated using the Mohr-Coulomb failure criterion. In direct relation to the results obtained at the face, tunnels aligned with the minor in-plane horizontal stress are far more sensitive to the use of specific failure criterion (difference in solid lines versus dashed lines in Figure 8-21). Overall, the Hoek-Brown failure criterion generates a yield profile that is 0.3 – 0.7 m larger in the crown and floor, as well as 0.1 – 0.3 m larger in the tunnel walls, as compared to the Mohr-Coulomb models. Note that the higher end values pertain to the tunnels aligned with the minor in-plane horizontal stress.

The results obtained from assessing a slice at the front of a blast round roughly three spans back from the face also suggest that the crown and floor are more sensitive to the failure criterion, compared to the tunnel walls (note lines in Figure 8-20 are more tightly grouped at the wall), when quantifying the extent of yielded rockmass and EDZ development.

8.2.4.3.2 Perimeter Tunnel Yield at a Blast Round Mid-Section

Modelling results pertaining to the extent of perimeter tunnel at the mid-section of a blast round roughly three spans back from the tunnel face again support the research presented by Hoek and Brown (1980) in terms of expected increase in yield at roughly three spans back from the face. Once again, the extent of increased yielded rockmass about the tunnel perimeter is predominantly related to the alignment of the tunnel axis with the in-plane horizontal stresses. Compared to an assessment at the tunnel face itself, the extent of EDZ development about the tunnel crown, floor and walls at the mid-section of a blast round roughly three spans back from
the face is up to 2.6 m larger for tunnels whose axes are aligned with the major in-plane horizontal stress, as compared to tunnels aligned with the minor in-plane horizontal stress.

With a slight variation of 0.1 – 0.4 m in extent of yielded rockmass, the results obtained from slices at the mid-section of a blast round roughly three spans back from the face present very similar perimeter yield profiles as those obtained from the front of a blast round (see correlation between Figure 8-21 and Figure 8-22).

Figure 8-22: (B): EDZ₀ development (m) about the tunnel perimeter at four blast rounds back from the tunnel face. (C): EDZ₀ development (m) about the tunnel perimeter at 4.5 blast rounds back from the tunnel face

Yet again, the varied use of failure criterion is noted to play a dramatic role in the resulting extent of EDZ₀ development about the tunnel perimeter. As in the case of the results
present in Figure 8-20, the Hoek-Brown failure criterion again results in a perimeter yield profile that is larger than that generated using the Mohr-Coulomb failure criterion (see Figure 8-22).

It was once more revealed that tunnels aligned with the minor in-plane horizontal stress are far more sensitive to the use of specific failure criterion. This phenomenon is highlighted but the spacing between the solid lines versus the dashed lines in Figure 8-22; note a generally larger spacing between the solid lines (tunnels aligned with the minor in-plane stress).

**8.2.4.4 Conclusions**

Based on the results presented above in Chapter 8.2.4, there does not appear to be a concise relationship between failure criterion and resulting yield ahead of an advancing tunnel face.

When assessing the yield about the tunnel perimeter, the Hoek-Brown failure criterion results in an EDZ profile that is larger than that generated using equivalent Mohr-Coulomb failure criterion. This conclusion holds true along the entire length of the tunnel and is irrespective of relative blast round location (face versus mid-section).

Similar to aforementioned results in Chapter 8.2.1, 8.2.2 and 8.2.3, the extent of EDZ development about the tunnel perimeter is again predominantly related to the alignment of the tunnel axis with the in-plane horizontal stresses.

Finally, results presented in Chapter 8.2.4 also suggest that when quantifying the extent of yielded rockmass and EDZ development, the crown, floor, and face, as compared to the tunnel walls, are more sensitive to the use of different failure criteria. Tunnels aligned with the minor in-plane horizontal stress are also slightly more sensitive to the use of specific failure criteria when assessing the perimeter yield profile, especially at the tunnel face.
8.3 Overall Conclusion

The aforementioned 3D FEM numerical modelling results presented in this chapter generally support the research presented by Hoek and Brown (1980) in the sense that the radius of yielded rockmass about the tunnel circumference/perimeter increases and reaches its maximum upon at roughly three spans back from the tunnel face. Furthermore, a slightly larger increase in yielded rockmass was generally observed in tunnels whose axes are aligned with the major in-plane horizontal stress.

Results presented in this chapter confirm that tunnel alignment per major and minor horizontal stresses is a vital factor in the extent of EDZ development about both the tunnel face and its perimeter/circumference. In direct support of this conclusion, continuously and conventional drill-and-blast excavated tunnels present the most significant contrast in results when the tunnel axis is aligned with the minor in-plane horizontal stress. Portions of unyielded rock in the tunnel walls were observed in tunnel whose axes are aligned with the minor in-plane horizontal stress. Additionally, 2D shape effects result in a more dramatic reduction of yield rockmass and size of the v-shaped notches when aligned with the orientation of maximum compressive stress. Furthermore, tunnels aligned with the minor in-plane horizontal stress are also slightly more sensitive to the use of specific failure criteria when assessing the perimeter yield profile, especially at the tunnel face.

Overall, it was determined that the blast round length does not play a significant role in the development of yielded rockmass ahead of an advancing tunnel face or around the perimeter.

Finally, assessment of the use of different failure criteria would suggest that Hoek-Brown models present greater EDZ development about the tunnel perimeter, regardless of in-plane horizontal stress; face yield results are less conclusive.
Chapter 9: A 3-Dimensional Assessment of The Impacts of Numerical Model Implementation of Brittle Rockmass Behaviour on Excavation Damage Zone Development

9.1 Introduction

Given the understanding of the discrepancies associated with modelling brittle rockmass behaviour in FEM versus FDM programs, this chapter presents a comparative assessment between results obtained using the DISL approach in FEM numerical models, compared to the CWFS approach used in FDM modelling.

9.1.1 Numerical Modelling: Input Parameters and Model Set-up

9.1.1.1 Material and Strength Properties

Refer to Chapter 2.4 for Cobourg Limestone material properties. Figure 2-6 in Chapter 2.3 displays the DISL input parameters for the Hoek-Brown and equivalent Mohr-Coulomb failure criteria.

9.1.1.2 Stresses and Depth

Each of the various models were again assessed at a depth of 700 mbgs. Refer to Table 8-1 in Chapter 8 for major and minor principal stresses that were calculated based on horizontal stress ratios (K values) of 1.5 and 3.5. Figure 8-1 in Chapter 8 presents a schematic of the insitu stress conditions and varying tunnel orientations also used in this chapter.

9.1.1.3 Numerical Modelling Software

RS3 version 1.0 was used to study the DISL modelling approach, while FLAC3D version 5.0 was used to assess the CWFS approach (see Chapter 2.2 for more information).
9.1.1.4 Mesh Discretization and Boundary Conditions

Each of the numerical models presented in Chapter 9 employed a square model boundary, 4x larger than the tunnel size, to reduce boundary effects. For the sake of increasing model efficiency, a half model (in comparison to a complete, full model used in RS³ – see Figure 8-2 in Chapter 8) was used to assess the tunnels in FLAC³D. The external boundaries were again pinned, while rollers were used on the external model plane that intersect the center line of the tunnel (ZY-plane), as well as the plane aligned with direction of tunnel advance (XZ-plane).

To maintain consistency in the size and gradation of the mesh elements used in the RS³ models of Chapter 8, the discretization density of the mesh elements in FLAC³D was increased around the excavation to employ a radially gradational model mesh, which aids to better delineate the extent of the EDZ around the tunnel perimeter. Note however, that unlike the RS³ models, a mesh boundary was not needed to employ this mesh formulation in the FLAC³D models. Figure 9-1 depicts these components of the FLAC³D FDM models. As per Walton and Diederichs (2015), the smallest mesh element measured no less than 3% of the tunnel radius.

Figure 9-1: 2D slice of FLAC³D – 3D FDM mesh (XZ-plane) for the rectangular (A) and circular (B) tunnel geometries.

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9.1.1.5 Model Geometry and Blast Round Length

For the purposes of this investigation, two tunnel shapes were assessed to enable a comparative analysis between the FEM and FDM results obtained using RS$^3$ and FLAC$^{3D}$, respectively; rectangular and circular geometries were assessed. Given the fact that the previous results presented in Chapter 8 that prove that blast round length does not play a significant role in EDZ development about the tunnel face nor perimeter, only 3.5 m blast round lengths were used for the modelling presented in this chapter (See Figure 9-2 below).

![Dimensions for the rectangular tunnel (left), and circular tunnel (right).](image)

Figure 9-2: Dimensions for the rectangular tunnel (left), and circular tunnel (right).

The length of the models in FLAC$^{3D}$ were again set to 6 times the tunnel span/diameter to properly capture stress flow and yield ahead of and behind the tunnel face. Note that only 60% of the tunnel was excavated to allow for the assessment and quantification of yield ahead of the tunnel face; this set-up is depicted in Figure 9-3 on the following page.
Figure 9-3: FLAC\textsuperscript{3D} – 3D FDM models depicting the rectangular (A) and circular (B) tunnel geometries.

For ease of comprehension, Table 9-1 highlights each of the key components of the various 3D models.

<table>
<thead>
<tr>
<th>Model</th>
<th>Geometry</th>
<th>Blast Round Length</th>
<th>$K \parallel$ to Tunnel</th>
<th>$K \perp$ to Tunnel</th>
<th>Failure Criterion</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Circle</td>
<td>3.5 m</td>
<td>1.5 x $\sigma_v$</td>
<td>3.5 x $\sigma_v$</td>
<td>DISL Mohr-Coulomb</td>
</tr>
<tr>
<td>2</td>
<td>Circle</td>
<td>3.5 m</td>
<td>3.5 x $\sigma_v$</td>
<td>1.5 x $\sigma_v$</td>
<td>DISL Mohr-Coulomb</td>
</tr>
<tr>
<td>3</td>
<td>Rectangle</td>
<td>3.5 m</td>
<td>1.5 x $\sigma_v$</td>
<td>3.5 x $\sigma_v$</td>
<td>DISL Mohr-Coulomb</td>
</tr>
<tr>
<td>4</td>
<td>Rectangle</td>
<td>3.5 m</td>
<td>3.5 x $\sigma_v$</td>
<td>1.5 x $\sigma_v$</td>
<td>DISL Mohr-Coulomb</td>
</tr>
<tr>
<td>5</td>
<td>Circle</td>
<td>3.5 m</td>
<td>1.5 x $\sigma_v$</td>
<td>3.5 x $\sigma_v$</td>
<td>CWFS Mohr-Coulomb</td>
</tr>
<tr>
<td>6</td>
<td>Circle</td>
<td>3.5 m</td>
<td>3.5 x $\sigma_v$</td>
<td>1.5 x $\sigma_v$</td>
<td>CWFS Mohr-Coulomb</td>
</tr>
<tr>
<td>7</td>
<td>Rectangle</td>
<td>3.5 m</td>
<td>1.5 x $\sigma_v$</td>
<td>3.5 x $\sigma_v$</td>
<td>CWFS Mohr-Coulomb</td>
</tr>
<tr>
<td>8</td>
<td>Rectangle</td>
<td>3.5 m</td>
<td>3.5 x $\sigma_v$</td>
<td>1.5 x $\sigma_v$</td>
<td>CWFS Mohr-Coulomb</td>
</tr>
</tbody>
</table>

Due to the complexities with modelling blast-induced damages, note that none of the modelling results in this chapter include such influence.
9.2 Results and Discussion

9.2.1 Yield Ahead of The Tunnel Face

Continuum numerical modelling results indicate that both the CWFS and DISL approaches to modelling brittle rockmass behaviour results in similar yield profiles ahead of an advancing tunnel face. It should be noted, however, that FLAC\textsuperscript{3D} FDM CWFS modelling results in slightly more yielded rockmass ahead of an advancing tunnel face as compared to equivalent RS\textsuperscript{3} FEM DISL models. A 0.2 m – 0.3 m discrepancy is evident in graph presented in Figure 9-4. Given the size of the mesh elements, this value can likely be partially attributed to mesh formulation.

![Figure 9-4: EDZ\textsubscript{o} development (m) about the tunnel face upon using different geometries and modelling programs (YZ-plane view is shown). Figure (A) depicts the circular tunnel profile, while Figure (B) depicts the rectangular profile.](image-url)
9.2.2 Circumferential/Perimeter Tunnel Yield at the Tunnel Face

Upon observing the yield profiles in Figure 9-5, it can be concluded that the use of the CWFS approach versus the DISL approach to modelling brittle rockmass behaviour results in yield profiles that although near-identical, may differ by up to 0.4 m. It should be noted that the yield profiles do not appear to be conclusively related to tunnel orientation, geometry or modelling approach.

![Diagram of Tunnel Face](image)

**Figure 9-5:** (A): EDZ₀ development (m) about the tunnel face circumference for the circular models. (B): EDZ₀ development (m) about the tunnel face perimeter for the rectangular models.
Of primary importance, however, is that near-identical results in terms of the EDZ perimeter yield at the tunnel face are indeed achieved, regardless of the method used to model brittle rockmass behaviour (denoted by the tight grouping of the lines in Figure 9-5).

9.2.3 Circumferential/Perimeter Tunnel Yield at Three Tunnel Spans Back from the Face

Similar to each of the previous investigations presented in Chapter 8, the perimeter yield about the tunnel geometries was assessed at roughly three tunnel spans back from the face. Since the models assessed in this chapter utilized 3.5 m blast rounds, the perimeter yield at the front of a blast round roughly three span back is located at 14 m (four blast rounds) back from the tunnel face. The slices used to assess the perimeter yield at the mid-section of a blast round is located at 15.75 m (4.5 blast rounds) back from the tunnel face. These locations are highlighted by the green and red lines, respectively, in Figure 9-6 and Figure 9-7.

Given the fact that one of the most striking conclusions presented in Chapter 7 pertained to the yield profile along the length of the tunnel, a similar comparison was conducted between results obtain from CWFS FDM modelling versus DISL FEM modelling.

As made evident by the images displayed in Figure 9-6 and Figure 9-7 on the following page, nearly identical yield profiles are generated regardless of the choice of modelling program or brittle damage model.
Figure 9-6: EDZ₀ development about the tunnel walls and face for the various circular models. Side view and tunnel advance direction shown. Yellow indicates yielded elements in RS³, while yield is displayed by various non-grey colours in FLAC³D. Refer to Figure 8-1 for colour coding. Left = tunnels aligned with K_h of 1.5. Right = tunnels aligned with K_H of 3.5. A 3.5 m blast round was used.

Figure 9-7: EDZ₀ development about the tunnel walls and face for the various rectangular models. Side view and tunnel advance direction shown. Yellow indicates yielded elements in RS³, while yield is displayed by various non-grey colours in FLAC³D. Left = tunnels aligned with K_h of 1.5. Right = tunnels aligned with K_H of 3.5. A 3.5 m blast round was used.
For tunnels aligned with the minor in-plane horizontal stress ($\sigma_2$), the walls of both models experience increased yield at each 3.5 m blast round increment (i.e. where the face is located after each blast round), while the intermediate wall sections present less radially yielded rockmass. This behaviour can be consistently observed in both the circular and rectangular geometries of Figure 9-6 and Figure 9-7, respectively.

Upon alignment of the tunnels with the major in-plane horizontal stress ($\sigma_1$), the shape of the yield profile was, however, noted to differ slightly. The walls of the FDM FLAC$^{3D}$ models that implement the CFWS approach were observed to contain a small baggage zone; this is denoted by the portion of unyielded (grey) in the walls of the FLAC$^{3D}$ models versus the continuous wall yield exhibited by the RS$^3$ DISL models.

### 9.2.3.1 Circumferential/Perimeter Tunnel Yield at the Front of a Blast Round

Recall that this location is highlighted by the green lines in Figure 9-6 and Figure 9-7 on the previous page.

Although the shape of EDZ in the tunnel walls is noted to be affected by the choice of CWFS versus DISL modelling for tunnels whose axes are aligned with the major in-plane horizontal stress, the overall extent of EDZ about the tunnels walls remains unaffected by the choice of modelling programs or brittle damage constitutive model. This is supported by the tight grouping of the dashed lines in Figure 9-8 on the following page.
Results presented in Figure 9-8 support the initial conclusion that nearly identical yield profiles are generated regardless of the choice of modelling program or brittle damage constitutive model. Tunnels aligned with the major in-plane horizontal stress (dashed lines in Figure 9-8) show a strong correlation and minimal discrepancy in resulting yield profile.

The shape and magnitude of the yield profiles generated by CWFS FDM FLAC\textsuperscript{3D} modelling versus DISL FEM RS\textsuperscript{3} modelling for tunnels whose axes are aligned with the minor in-plane horizontal stress differ by up to 0.4 m. This is primarily noted to occur at the corners of the yield profiles rather than close to the excavation boundary. This can be attributed to the formulation of the two modelling approaches. As discussed in Chapter 2.2, FEM models use two
distinct curves to model brittle rockmass behaviour. Additionally, the transition between DISL peak and residual cohesive and frictional strength components considered effectively instantaneous. However, FDM models incorporate a strain-dependency factor that allows the user to define the point at which the cohesive and frictional strength components are mobilized; FDM CWFS models are therefore transitional state models. Henceforth, the fact that the two modelling approaches handle confinement in different manners largely impacts the resulting extent of EDZ development at distances further away from the excavation boundary.

This conclusion therefore highlights the key finding of Chapter 9; the choice of modelling program and brittle damage model plays a more significant role in the resulting yield propagation and eventual termination (formation of the EDZ and EIZ boundary) as a result in differing stress paths and confinement brought on by the use of CWFS versus DISL brittle damage model.

9.2.3.2 Circumferential/Perimeter Tunnel Yield at the Mid-Section of a Blast Round

Recall that this location is highlighted by the red lines in Figure 9-6 and Figure 9-7.

Quantitative comparisons of the results presented in Figure 9-9 present a strong correlation to the aforementioned results obtained from the face of a blast round. The shape and extent of the yield profiles generated by CWFS FLAC\textsuperscript{3D} models versus DISL RS\textsuperscript{3} models, for tunnels whose axes are aligned with the minor in-plane horizontal stress, present the greatest discrepancy at the corners of the yield profiles rather than close to the excavation boundary. This is supported by the spacing in the solid lines, as compared to the tighter grouping of the dashes lines.
Figure 9-9: (A): EDZo development (m) about the circular tunnel perimeters at 4.5 blast rounds back from the tunnel face. (B): EDZo development (m) about the rectangular tunnel perimeters at 4.5 blast rounds back from the tunnel face.

9.3 Conclusions

3D FEM and FDM numerical modelling results presented in this chapter highlight several key findings of the comparative assessment of EDZ development resulting from CWFS versus DISL modelling of brittle rockmass damage.

It was determined that although both the CWFS and DISL approaches result in similar yield profiles ahead of an advancing tunnel face, FLAC\textsuperscript{3D} CWFS modelling generates slightly more yielded rockmass as compared to equivalent RS\textsuperscript{3} DISL models. Results of the perimeter yield profiles also correlate to this conclusion in the sense that although minor discrepancies were
noted, near-identical results in terms of the perimeter yield at the funnel face are indeed achieved upon using either CWFS or DISL brittle damage models.

Portions of unyielded rock in the tunnel walls were observed in tunnels whose axes are aligned with the minor in-plane horizontal stress for both brittle damage modelling approaches. Upon assessing tunnels aligned with the major in-plane horizontal stress, the walls of the FLAC$^3$D models that implement the CFWS approach were observed to contain a small baggage zone, while continuous wall yield was exhibited by the RS$^3$ DISL models. Although the wall yield profile is noted to be affected by tunnel alignment and the choice of CWFS versus DISL modelling, the overall radial extent of EDZ development about the tunnels walls is independent of modelling programs or brittle damage constitutive model.

Finally, due to the disparity in post-yield behaviour and resulting stress paths and confinement, the shape and magnitude of the EDZ profile generated by FLAC$^3$D CWFS models versus RS$^3$ DISL models are noted to present differing results at the corners of the yield profiles for tunnels whose axes are aligned with the minor in-plane horizontal stress.
Chapter 10: Conclusions

10.1 Overview of Thesis Objective

The prediction and understanding of the EDZ development around DGR placement room tunnels and storage voids is a vital component of design for ensuring safe, long-term disposal and containment of high-level nuclear waste. The choice of macro-geometry details, such as tunnel shape, profile and corner details, are often made for logistical or ground support reasons for the operational phase. These decisions will also impact the ultimate development of EDZ and therefore the long-term safety performance of the repository.

With the use of various 2-dimensional and 3-dimensional continuum numerical modelling simulations, this thesis has illustrated various key impacts of such macro-geometry choices, as well as implications associated with excavation sequence and methodology on brittle damage development around deep underground excavations. This thesis also highlights the fact that discrepancies in numerical simulations can arise as a result of the choice of failure criteria, brittle damage model or numerical modelling software.

The findings presented in this thesis will ultimately contribute to the determination of safe and economic design details for future underground nuclear waste placement rooms and storage tunnels.

10.2 Summary of Work

This thesis presents a comprehensive study of continuum modelling implications associated with predicting and categorizing EDZ development around deep underground excavations within brittle rockmasses. Various comparative assessments were conducted by using 2-dimensional FEM and 3-dimensional FEM and FDM numerical modelling software.

RS2, a 2D FEM numerical modelling program, was used to study the impact of macro-geometry choices (shape, profile and corners details) on resulting brittle damage development and
EDZ categorization, as well as for a comparative study of brittle damage resulting from the varied use of Hoek-Brown DISL versus equivalent Mohr-Coulomb DISL failure criteria.

Rocscience’s RS² and RS³ were then used to study the impact of 2-dimensional versus 3-dimensional FEM numerical modelling on the resulting magnitude and extent of brittle damage development around deep underground excavations.

A 3-dimensional FEM study using RS³ was subsequently carried out in order to complete a multi-faceted, interconnected assessment of the impacts of tunnel shape/profile, orientation, blast round length and failure criteria on the resulting EDZ development and yield profile.

The final analysis of this thesis presents a comparative assessment of the excavation damage zone and yield profile that results when using RocScience’s RS³ versus Itasca’s FLAC³D. This study was completed in order to quantify to impact of using different constitutive brittle damage approaches when modelling EDZ development about deep underground excavations.

10.3 Final Conclusions

Upon studying the impact of macro-geometry choices on resulting brittle damage development and EDZ categorization, it is evident that several issues and challenges arise when modelling brittle EDZ evolution with a 2D continuum model. Results presented in Chapter 3 reveal that the choice of DISL brittle damage model (Hoek-Brown versus equivalent Mohr-Coulomb) impacts the nature and extent of EDZ for moderate stress states. 2-dimensional FEM modelling proves that corners and flat boundaries create potentially undamaged baggage zones that have also been observed in mining tunnels. The existence of the baggage zone raises the question as to its inclusion within the HDZ when categorizing EDZ for nuclear waste disposal purposes. Overall, the generalized guidelines for 2D FEM model interpretation need to be revised to encompass corner and baggage issues.
Results presented in Chapter 5 2D FEM comparative study of brittle damage resulting from the varied use of Hoek-Brown DISL versus equivalent Mohr-Coulomb DISL failure criteria indicate that as the tunnel aspect ratio diverges from 1:1 (and becomes more rectangular in shape), the tunnel is more subjective to the use of different failure criteria at lower stress states. As the aspect ratio approaches 1, 2D FEM models appear to be far less sensitive to the horizontal stress ratio when modelling with either failure criteria. Results discussed in Chapter 5 also globally indicate that for insitu stress regimes with a horizontal stress ratio greater than or equal to 1.75, the EDZ development about an underground excavation, of any shape, are independent of the failure criteria.

2D FEM numerical modelling results presented in Chapter 3 and Chapter 5 additionally prove that the existence, profile and extent of the baggage zone is dependent on both the stress regime and the failure criteria. Results presented in Chapter 6 also prove that corner details play a significant role in the resulting reduction in baggage zone profile and extent, as well as the dissipation of tensile failure in the tunnel walls.

Chapter 6 highlights numerous issues and challenges encountered when modelling brittle EDZ evolution with 2D versus 3D FEM continuum models. 3D modelling shows that in some cases, the stress changes around the advancing tunnel face create a continuous brittle damage zone, while baggage exists only in the equivalent 2D simulation.

Results discussed in Chapter 8 generally support the research presented by Hoek and Brown (1980) in the sense that the radius of yielded rockmass about the tunnel perimeter increases and reaches its maximum upon at roughly three tunnel spans back from the face. 3D FEM modelling results confirm that tunnel alignment relative to major and minor horizontal stresses is a vital factor in the extent of EDZ development about both the tunnel face and its perimeter. The choice of conventional drill-and-blast versus continuous excavation results in dramatically different yield profiles along the length of the tunnel walls; this phenomenon is
amplified in tunnels whose axes are aligned with the minor in-plane horizontal stress. This result can also be attributed to the geometrical choices; the continuous excavation employs a circular tunnel to represent a TBM profile, while the drill-and-blast excavation is conducted with a non-circular tunnel profile. Additionally, 2D shape effects result in a more dramatic reduction of yield rockmass and size of the v-shaped notches when aligned with the orientation of maximum compressive stress. It was concluded that blast round length does not play a significant role in the development of yielded rockmass ahead of an advancing tunnel face or around the perimeter. Results presented in Chapter 8 would also suggest that Hoek-Brown models present greater EDZ development about the tunnel perimeter as compared to equivalent Mohr-Coulomb models, regardless of the in-plane horizontal stress.

Based on the results discussed in Chapter 9, it was as determined that near-identical EDZ development and yield profiles at the tunnel face are indeed achieved upon using either CWFS or DISL brittle damage approaches, despite minor discrepancies. While the tunnel wall yield profile was noted to be affected by tunnel alignment and the choice of CWFS versus DISL, the overall radial extent of EDZ development has been proven independent of modelling program or brittle damage model. One of the key findings presented in Chapter 9 highlights the fact that the variation in post-yield behaviour and resulting stress paths and confinement in FLAC$^{\text{3D}}$ CWFS models versus RS$^{\text{3}}$ DISL models leads to variable EDZ development and differing final yield profile about the perimeter of deep underground excavations in brittle rock.

10.4 Future Work

While this thesis presents a comprehensive study of macro-geometry choice details and their impacts on 2D and 3D numerical modelling of EDZ development in brittle rock, there are several topics that may benefit from future investigations.
Due to the complexity of modelling blast-induced damage in FEM and FDM software, none of the numerical models analyzed in this thesis include such affects. To further quantity and draw more realistic comparisons between the impacts of continuous versus conventional drill-and-blast excavation techniques, incorporating blast-induced-damage in future numerical models is suggested. This would also aid in developing a greater understand of the impacts of blast round length on both EDZ development ahead of an advancing tunnel face, as well as along the overall length of the tunnel.

Since the future high-level DGR will contain a vast network of placement room and access tunnels, in addition to several vertical shafts, an understanding of the implications of tunnel intersections on resulting EDZ development is paramount to the long-term safety of the DGR project. In order to establish an understanding of such geometrically-induced interactions, it is suggested that future numerical modelling includes the simulation of tunnel intersections, as well as parallel, side-by-side placement room tunnels.

It has been proven that EDZ cut-off seals can dramatically impact the nature and extent of fracture propagation along DGR vertical shafts (Perras et al. 2015). In continuation with the cut-off research program implemented by the NWMO in 2014, numerical modelling of these key-shaped cut-off seals within placement room tunnels would aid in quantifying the reduction in fracture propagation and EDZ development along the tunnel axes.

In order to develop a better understanding of brittle fracture mechanics, finite-discrete element method (FDEM) could be used in future modelling. Programs such as Geomechanica’s Irazu (Geomechanica Inc. 2016) “combines continuum mechanics principles with discrete element method (DEM) algorithms to simulate multiple interacting deformable bodies”. By using this FDEM software, the user can visually assess and quantify the magnitude and extent of brittle fractures that propagate through the model, which would ultimately aid in developing a better understanding of EDZ development.
In addition to the aforementioned avenues that could be used to build on the scope of numerical modelling presented in this thesis, modelling input parameters should be updated once a specific APM DGR site is selected by the NWMO. This would serve to better represent the true, expected, insitu rockmass and stress conditions. Updating the model input parameters will ultimately generate more accurate modelling outputs that will aid in better quantifying the predicted EDZ development about the DGR placement room and access tunnels.
Appendix A: List of Models and Input Parameters

Chapter 4 Numerical Models

Below is a list of the 12 numerical models used in the analyses presented in Chapter 4.

Table A - 1: List of numerical models presented in Chapter 4.

| Model | Program | Geometry       | Excavation Type          | K || to Tunnel | K | to Tunnel | Failure Criterion |
|-------|---------|----------------|--------------------------|---------------|----------------|-----------------|------------------|
| 1     | RS²     | Rectangle      | Staged Excavation        | 1.25 x σv     | 1.5 x σv       | DISL H-B        |
| 2     | RS²     | Horseshoe      | Staged Excavation        | 1.25 x σv     | 1.5 x σv       | DISL H-B        |
| 3     | RS²     | Rounded Corners| Staged Excavation        | 1.25 x σv     | 1.5 x σv       | DISL H-B        |
| 4     | RS²     | Rectangle      | Staged Excavation        | 1.25 x σv     | 1.5 x σv       | DISL M-C        |
| 5     | RS²     | Horseshoe      | Staged Excavation        | 1.25 x σv     | 1.5 x σv       | DISL M-C        |
| 6     | RS²     | Rounded Corners| Staged Excavation        | 1.25 x σv     | 1.5 x σv       | DISL M-C        |
| 7     | RS²     | Rectangle      | Staged Excavation        | 1.25 x σv     | 1.5 x σv       | DISL H-B        |
| 8     | RS²     | Horseshoe      | Staged Excavation        | 1.5 x σv      | 2.0 x σv       | DISL H-B        |
| 9     | RS²     | Rounded Corners| Staged Excavation        | 1.5 x σv      | 2.0 x σv       | DISL H-B        |
| 10    | RS²     | Rectangle      | Staged Excavation        | 1.5 x σv      | 2.0 x σv       | DISL M-C        |
| 11    | RS²     | Horseshoe      | Staged Excavation        | 1.5 x σv      | 2.0 x σv       | DISL M-C        |
| 12    | RS²     | Rounded Corners| Staged Excavation        | 1.5 x σv      | 2.0 x σv       | DISL M-C        |
Chapter 5 Numerical Models

Below is a list of the 84 numerical models used in the analyses presented in Chapter 5.

Table A - 2: List of numerical models presented in Chapter 5.

<table>
<thead>
<tr>
<th>Model</th>
<th>Program</th>
<th>Geometry</th>
<th>Excavation Type</th>
<th>$K \parallel$ to Tunnel</th>
<th>$K \parallel$ to Tunnel</th>
<th>Failure Criterion</th>
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<td>1.5 x $\sigma_v$</td>
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</table>
Chapter 6 Numerical Models

Below is a list of the 56 numerical models used in the analyses presented in Chapter 6.

Table A - 3: List of numerical models presented in Chapter 6.

<p>| Model | Program | Geometry       | Excavation Type | K || to Tunnel | K | to Tunnel | Failure Criterion |
|-------|---------|----------------|-----------------|---------------|---------------|---------------|-------------------|
| 1     | RS²     | Square         | Staged Excavation | 1.25 x σv     | 1.5 x σv     | DISL H-B      |
| 2     | RS²     | Square         | Staged Excavation | 1.25 x σv     | 1.5 x σv     | DISL M-C      |
| 3     | RS²     | Rounded 0.25 m | Staged Excavation | 1.25 x σv     | 1.5 x σv     | DISL H-B      |
| 4     | RS²     | Rounded 0.25 m | Staged Excavation | 1.25 x σv     | 1.5 x σv     | DISL M-C      |
| 5     | RS²     | Rounded 0.5 m  | Staged Excavation | 1.25 x σv     | 1.5 x σv     | DISL H-B      |
| 6     | RS²     | Rounded 0.5 m  | Staged Excavation | 1.25 x σv     | 1.5 x σv     | DISL M-C      |
| 7     | RS²     | Rounded 0.75 m | Staged Excavation | 1.25 x σv     | 1.5 x σv     | DISL H-B      |
| 8     | RS²     | Rounded 0.75 m | Staged Excavation | 1.25 x σv     | 1.5 x σv     | DISL M-C      |
| 9     | RS²     | Rounded 1.0 m  | Staged Excavation | 1.25 x σv     | 1.5 x σv     | DISL H-B      |
| 10    | RS²     | Rounded 1.0 m  | Staged Excavation | 1.25 x σv     | 1.5 x σv     | DISL M-C      |
| 11    | RS²     | Rounded 1.25 m | Staged Excavation | 1.25 x σv     | 1.5 x σv     | DISL H-B      |
| 12    | RS²     | Rounded 1.25 m | Staged Excavation | 1.25 x σv     | 1.5 x σv     | DISL M-C      |
| 13    | RS²     | Circle         | Staged Excavation | 1.25 x σv     | 1.5 x σv     | DISL H-B      |
| 14    | RS²     | Circle         | Staged Excavation | 1.25 x σv     | 1.5 x σv     | DISL M-C      |
| 15    | RS²     | Square         | Staged Excavation | 1.375 x σv     | 1.75 x σv     | DISL H-B      |
| 16    | RS²     | Square         | Staged Excavation | 1.375 x σv     | 1.75 x σv     | DISL M-C      |
| 17    | RS²     | Rounded 0.25 m | Staged Excavation | 1.375 x σv     | 1.75 x σv     | DISL H-B      |
| 18    | RS²     | Rounded 0.25 m | Staged Excavation | 1.375 x σv     | 1.75 x σv     | DISL M-C      |
| 19    | RS²     | Rounded 0.5 m  | Staged Excavation | 1.375 x σv     | 1.75 x σv     | DISL H-B      |
| 20    | RS²     | Rounded 0.5 m  | Staged Excavation | 1.375 x σv     | 1.75 x σv     | DISL M-C      |
| 21    | RS²     | Rounded 0.75 m | Staged Excavation | 1.375 x σv     | 1.75 x σv     | DISL H-B      |
| 22    | RS²     | Rounded 0.75 m | Staged Excavation | 1.375 x σv     | 1.75 x σv     | DISL M-C      |
| 23    | RS²     | Rounded 1.0 m  | Staged Excavation | 1.375 x σv     | 1.75 x σv     | DISL H-B      |
| 24    | RS²     | Rounded 1.0 m  | Staged Excavation | 1.375 x σv     | 1.75 x σv     | DISL M-C      |
| 25    | RS²     | Rounded 1.25 m | Staged Excavation | 1.375 x σv     | 1.75 x σv     | DISL H-B      |
| 26    | RS²     | Rounded 1.25 m | Staged Excavation | 1.375 x σv     | 1.75 x σv     | DISL M-C      |
| 27    | RS²     | Circle         | Staged Excavation | 1.375 x σv     | 1.75 x σv     | DISL H-B      |
| 28    | RS²     | Circle         | Staged Excavation | 1.375 x σv     | 1.75 x σv     | DISL M-C      |
| 29    | RS²     | Square         | Staged Excavation | 1.5 x σv       | 2.0 x σv     | DISL H-B      |
| 30    | RS²     | Square         | Staged Excavation | 1.5 x σv       | 2.0 x σv     | DISL M-C      |
| 31    | RS²     | Rounded 0.25 m | Staged Excavation | 1.5 x σv       | 2.0 x σv     | DISL H-B      |
| 32    | RS²     | Rounded 0.25 m | Staged Excavation | 1.5 x σv       | 2.0 x σv     | DISL M-C      |
| 33    | RS²     | Rounded 0.5 m  | Staged Excavation | 1.5 x σv       | 2.0 x σv     | DISL H-B      |</p>
<table>
<thead>
<tr>
<th></th>
<th>RS²</th>
<th>Shape</th>
<th>Excavation Method</th>
<th>Multiplier</th>
<th>Multiplier</th>
<th>Dislocation</th>
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<td>Staged Excavation</td>
<td>1.5 x σ_v</td>
<td>2.0 x σ_v</td>
<td>DISL M-C</td>
</tr>
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<td>Staged Excavation</td>
<td>1.5 x σ_v</td>
<td>2.0 x σ_v</td>
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</tr>
<tr>
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<td>Rounded 0.75 m</td>
<td>Staged Excavation</td>
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<td>2.0 x σ_v</td>
<td>DISL M-C</td>
</tr>
<tr>
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<td>Staged Excavation</td>
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<td>2.0 x σ_v</td>
<td>DISL H-B</td>
</tr>
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<td>2.0 x σ_v</td>
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<td>2.0 x σ_v</td>
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<td>2.5 x σ_v</td>
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<td>2.5 x σ_v</td>
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<td>2.5 x σ_v</td>
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<td>2.5 x σ_v</td>
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<td>2.5 x σ_v</td>
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<td>Staged Excavation</td>
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<td>2.5 x σ_v</td>
<td>DISL M-C</td>
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<td>2.5 x σ_v</td>
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<td>2.5 x σ_v</td>
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</tr>
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<td>2.5 x σ_v</td>
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<td>2.5 x σ_v</td>
<td>DISL M-C</td>
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<td>RS²</td>
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<td>Staged Excavation</td>
<td>1.75 x σ_v</td>
<td>2.5 x σ_v</td>
<td>DISL H-B</td>
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<td>DISL M-C</td>
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<td>1.75 x σ_v</td>
<td>2.5 x σ_v</td>
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</table>
Chapter 7 Numerical Models

Below is a list of the 9 numerical models used in the analyses presented in Chapter 7.

Table A - 4: List of numerical models presented in Chapter 7.

| Model | Program | Geometry    | Excavation Type | $K_{||}$ to Tunnel | $K_{\perp}$ to Tunnel | Failure Criterion |
|-------|---------|-------------|-----------------|--------------------|------------------------|-------------------|
| 1     | RS$^2$  | Rectangle   | Instant Excavation | 1.5 $\times$ $\sigma_v$ | 2.0 $\times$ $\sigma_v$ | DISL H-B          |
| 2     | RS$^2$  | Horseshoe   | Instant Excavation | 1.5 $\times$ $\sigma_v$ | 2.0 $\times$ $\sigma_v$ | DISL H-B          |
| 3     | RS$^2$  | Rounded Corners | Instant Excavation | 1.5 $\times$ $\sigma_v$ | 2.0 $\times$ $\sigma_v$ | DISL H-B          |
| 4     | RS$^2$  | Rectangle   | Staged Excavation | 1.5 $\times$ $\sigma_v$ | 2.0 $\times$ $\sigma_v$ | DISL H-B          |
| 5     | RS$^2$  | Horseshoe   | Staged Excavation | 1.5 $\times$ $\sigma_v$ | 2.0 $\times$ $\sigma_v$ | DISL H-B          |
| 6     | RS$^2$  | Rounded Corners | Staged Excavation | 1.5 $\times$ $\sigma_v$ | 2.0 $\times$ $\sigma_v$ | DISL H-B          |
| 7     | RS$^3$  | Rectangle   | Staged Excavation | 1.5 $\times$ $\sigma_v$ | 2.0 $\times$ $\sigma_v$ | DISL H-B          |
| 8     | RS$^3$  | Horseshoe   | Staged Excavation | 1.5 $\times$ $\sigma_v$ | 2.0 $\times$ $\sigma_v$ | DISL H-B          |
| 9     | RS$^3$  | Rounded Corners | Staged Excavation | 1.5 $\times$ $\sigma_v$ | 2.0 $\times$ $\sigma_v$ | DISL H-B          |
Chapter 8 Numerical Models

Below is a list of the 14 numerical models used in the analyses presented in Chapter 8.

Table A - 5: List of numerical models presented in Chapter 8.

<table>
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<tr>
<th>Model</th>
<th>Program</th>
<th>Geometry</th>
<th>Blast Round Length</th>
<th>K // to Tunnel</th>
<th>K | to Tunnel</th>
<th>Failure Criterion</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>RS³</td>
<td>Circle</td>
<td>Continuous*</td>
<td>1.5 x (\sigma_v)</td>
<td>3.5 x (\sigma_v)</td>
<td>DISL H-B</td>
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<td>2</td>
<td>RS³</td>
<td>Circle</td>
<td>Continuous*</td>
<td>3.5 x (\sigma_v)</td>
<td>1.5 x (\sigma_v)</td>
<td>DISL H-B</td>
</tr>
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<td>3</td>
<td>RS³</td>
<td>Rectangle</td>
<td>2.5 m</td>
<td>1.5 x (\sigma_v)</td>
<td>3.5 x (\sigma_v)</td>
<td>DISL H-B</td>
</tr>
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<td>RS³</td>
<td>Rectangle</td>
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<td>3.5 x (\sigma_v)</td>
<td>1.5 x (\sigma_v)</td>
<td>DISL H-B</td>
</tr>
<tr>
<td>5</td>
<td>RS³</td>
<td>Rounded Corners</td>
<td>2.5 m</td>
<td>1.5 x (\sigma_v)</td>
<td>3.5 x (\sigma_v)</td>
<td>DISL H-B</td>
</tr>
<tr>
<td>6</td>
<td>RS³</td>
<td>Rounded Corners</td>
<td>2.5 m</td>
<td>3.5 x (\sigma_v)</td>
<td>1.5 x (\sigma_v)</td>
<td>DISL H-B</td>
</tr>
<tr>
<td>7</td>
<td>RS³</td>
<td>Horseshoe</td>
<td>2.5 m</td>
<td>1.5 x (\sigma_v)</td>
<td>3.5 x (\sigma_v)</td>
<td>DISL H-B</td>
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<tr>
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<td>RS³</td>
<td>Horseshoe</td>
<td>2.5 m</td>
<td>3.5 x (\sigma_v)</td>
<td>1.5 x (\sigma_v)</td>
<td>DISL H-B</td>
</tr>
<tr>
<td>9</td>
<td>RS³</td>
<td>Horseshoe</td>
<td>1.5 m</td>
<td>1.5 x (\sigma_v)</td>
<td>3.5 x (\sigma_v)</td>
<td>DISL H-B</td>
</tr>
<tr>
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<td>RS³</td>
<td>Horseshoe</td>
<td>1.5 m</td>
<td>3.5 x (\sigma_v)</td>
<td>1.5 x (\sigma_v)</td>
<td>DISL H-B</td>
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<td>3.5 m</td>
<td>1.5 x (\sigma_v)</td>
<td>3.5 x (\sigma_v)</td>
<td>DISL H-B</td>
</tr>
<tr>
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<td>Horseshoe</td>
<td>3.5 m</td>
<td>3.5 x (\sigma_v)</td>
<td>1.5 x (\sigma_v)</td>
<td>DISL H-B</td>
</tr>
<tr>
<td>13</td>
<td>RS³</td>
<td>Horseshoe</td>
<td>3.5 m</td>
<td>1.5 x (\sigma_v)</td>
<td>3.5 x (\sigma_v)</td>
<td>DISL M-C</td>
</tr>
<tr>
<td>14</td>
<td>RS³</td>
<td>Horseshoe</td>
<td>3.5 m</td>
<td>3.5 x (\sigma_v)</td>
<td>1.5 x (\sigma_v)</td>
<td>DISL M-C</td>
</tr>
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</table>
Chapter 9 Numerical Models

Below is a list of the 8 numerical models used in the analyses presented in Chapter 9.

Table A - 6: List of numerical models presented in Chapter 9.

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<thead>
<tr>
<th>Model</th>
<th>Program</th>
<th>Geometry</th>
<th>Blast Round Length</th>
<th>$K \parallel$ to Tunnel</th>
<th>$K \perp$ to Tunnel</th>
<th>Failure Criterion</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>RS$^3$</td>
<td>Circle</td>
<td>3.5 m</td>
<td>1.5 $\times$ $\sigma_v$</td>
<td>3.5 $\times$ $\sigma_v$</td>
<td>DISL M-C</td>
</tr>
<tr>
<td>2</td>
<td>RS$^3$</td>
<td>Circle</td>
<td>3.5 m</td>
<td>3.5 $\times$ $\sigma_v$</td>
<td>1.5 $\times$ $\sigma_v$</td>
<td>DISL M-C</td>
</tr>
<tr>
<td>3</td>
<td>RS$^3$</td>
<td>Rectangle</td>
<td>3.5 m</td>
<td>1.5 $\times$ $\sigma_v$</td>
<td>3.5 $\times$ $\sigma_v$</td>
<td>DISL M-C</td>
</tr>
<tr>
<td>4</td>
<td>RS$^3$</td>
<td>Rectangle</td>
<td>3.5 m</td>
<td>3.5 $\times$ $\sigma_v$</td>
<td>1.5 $\times$ $\sigma_v$</td>
<td>DISL M-C</td>
</tr>
<tr>
<td>5</td>
<td>FLAC$^{3D}$</td>
<td>Circle</td>
<td>3.5 m</td>
<td>1.5 $\times$ $\sigma_v$</td>
<td>3.5 $\times$ $\sigma_v$</td>
<td>CWFS M-C</td>
</tr>
<tr>
<td>6</td>
<td>FLAC$^{3D}$</td>
<td>Circle</td>
<td>3.5 m</td>
<td>3.5 $\times$ $\sigma_v$</td>
<td>1.5 $\times$ $\sigma_v$</td>
<td>CWFS M-C</td>
</tr>
<tr>
<td>7</td>
<td>FLAC$^{3D}$</td>
<td>Rectangle</td>
<td>3.5 m</td>
<td>1.5 $\times$ $\sigma_v$</td>
<td>3.5 $\times$ $\sigma_v$</td>
<td>CWFS M-C</td>
</tr>
<tr>
<td>8</td>
<td>FLAC$^{3D}$</td>
<td>Rectangle</td>
<td>3.5 m</td>
<td>3.5 $\times$ $\sigma_v$</td>
<td>1.5 $\times$ $\sigma_v$</td>
<td>CWFS M-C</td>
</tr>
</tbody>
</table>
Appendix B: Elastic and Plastic Solution Verification

To ensure that the modelling software was function properly, both elastic and plastic solution verifications were conducted. The Kirsch Solution (Kirsch 1898) was used for the elastic verification of the various RS², RS³ and FLAC³D models, while the closed form solution was used for the plastic model verification of each of aforementioned software.

Elastic Solution Verification – Kirsch Solution

Both hydrostatic and anisotropic stress conditions were verified with the Kirsch Solution. Note that the horizontal stress ratios ("K Factor") assessed in the following figures correspond to the values used in the models discussed in this thesis. The elastic Kirsch Solution verification can be used for both Mohr-Coulomb and Hoek-Brown failure criteria.

Plastic Verification – Closed Form Solution

Given the limited applicability of the plastic Closed Form Solution, only hydrostatic stress conditions and Hoek-Brown failure criterion were evaluated. Again, this verification was applied to RS², RS³ and FLAC³D models.
Appendix B - 1: Elastic Kirsch Solution RS$^2$ verification for a 2.5 m diameter tunnel under hydrostatic stress conditions using Mohr-Coulomb failure criterion.
Appendix B - 2: Elastic Kirsch solution $RS^2$ verification for a 2.5 m diameter tunnel under hydrostatic stress conditions using Hoek-Brown failure criterion.

<table>
<thead>
<tr>
<th>Kirsch Solution for Biaxial Field Stress – $RS^2$</th>
<th>Simplifications of Kirsch Solution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical Stress: 18.0 MPa</td>
<td>$\sigma_{1f}$</td>
</tr>
<tr>
<td>K-Factor: 1.0 (horizontal/vertical stress)</td>
<td>$\sigma_{2f}$</td>
</tr>
<tr>
<td>Failure Criterion: Hoek-Brown</td>
<td>$\sigma_{3f}$</td>
</tr>
<tr>
<td>Tunnel Radius: 2.5 m</td>
<td>$3\sigma_{1f}$</td>
</tr>
</tbody>
</table>

![Graph showing stress distribution around a tunnel](image)

- Model $\sigma_3$ Stress
- Model $\sigma_1$ Stress
- Kirsch Solution Radial Stress
- Kirsch Solution Tangential Stress

TUNNEL
Appendix B - 3: Elastic Kirsch solution RS² verification for a 2.5 m diameter tunnel, using a horizontal stress ratio (“K Factor”) or 1.5 and Hoek-Brown failure criterion.
Appendix B - 4: Elastic Kirsch solution RS² verification for a 2.5 m diameter tunnel, using a horizontal stress ratio (“K Factor”) of 2.0 and Hoek-Brown failure criterion.
Appendix B - 5: Elastic Kirsch solution $RS^2$ verification for a 2.5 m diameter tunnel, using a horizontal stress ratio (“K Factor”) or 2.5 and Hoek-Brown failure criterion.
Appendix B - 6: Elastic Kirsch Solution RS³ verification for a 2.5 m diameter tunnel under hydrostatic stress conditions using Mohr-Coulomb failure criterion.
### Appendix B - 7: Elastic Kirsch solution RS³ verification for a 2.5 m diameter tunnel under hydrostatic stress conditions using Hoek-Brown failure criterion.

<table>
<thead>
<tr>
<th>Kirsch Solution for Biaxial Field Stress – RS³</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical Stress: 18.9 MPa</td>
</tr>
<tr>
<td>K Factor: 1.0 (horizontal/vertical stress)</td>
</tr>
<tr>
<td>Failure Criterion: Hoek-Brown</td>
</tr>
<tr>
<td>Tunnel Radius: 2.5 m</td>
</tr>
</tbody>
</table>

![Diagram of Kirsch Solution RS³ Verification](image)

- Model $\sigma_3$ Stress
- Model $\sigma_1$ Stress
- Kirsch Solution Radial Stress
- Kirsch Solution Tangential Stress

TUNNEL 2.5m

Distance (m) vs. Stress (MPa) graph showing the comparison of stresses around the tunnel.
Appendix B - 8: Elastic Kirsch solution RS³ verification for a 2.5 m diameter tunnel, using a horizontal stress ratio (“K Factor”) or 1.5 and Hoek-Brown failure criterion.
Appendix B - 9: Elastic Kirsch solution RS⁵ verification for a 2.5 m diameter tunnel, using a horizontal stress ratio ("K Factor") or 3.5 and Hoek-Brown failure criterion.
Appendix B - 10: Elastic Kirsch Solution FLAC$^{3D}$ verification for a 2.5 m diameter tunnel under hydrostatic stress conditions using Mohr-Coulomb failure criterion.
Appendix B - 11: Elastic Kirsch solution FLAC$^{3D}$ verification for a 2.5 m diameter tunnel, using a horizontal stress ratio (“K Factor”) or 1.5 and Mohr-Coulomb failure criterion.
Appendix B - 12: Elastic Kirsch solution FLAC$^{3D}$ verification for a 2.5 m diameter tunnel, using a horizontal stress ratio ("K Factor") of 3.5 and Mohr-Coulomb failure criterion.
Appendix B - 13: Plastic Closed Form Solution verification for a 2.5 m diameter tunnel under hydrostatic stress conditions using Hoek-Brown failure criterion.
Appendix B - 14: Plastic Closed Form Solution verification for a 2.5 m diameter tunnel under hydrostatic stress conditions using Hoek-Brown failure criterion.
Appendix B - 15: Plastic Closed Form Solution verification for a 2.5 m diameter tunnel under hydrostatic stress conditions using Hoek-Brown failure criterion.
Appendix C: Validation of Numerical Models

Boundary Conditions

As previously stated in each of the numerical modelling chapters presented in this thesis, a square model boundary five times larger than the tunnel span/diameter was used for the 2D RS\textsuperscript{2} models. Pins were applied to each of the external boundaries of the RS\textsuperscript{2} models.

Given the fact that the 3D RS\textsuperscript{3} models are computationally heavier and require a far longer time to solve as compared to the RS\textsuperscript{2} models, a square model boundary four times larger than the tunnel span/diameter was used for the RS\textsuperscript{3} models. Rollers were applied to the two parallel planes within which the tunnel intersected (XY-plane), while pins were applied to the four remaining external boundaries.

In an effort to further save on computation time, the FLAC\textsuperscript{3D} tunnel excavations were simulated using a half model. Note that the square model boundary was still selected to measure four times that of the tunnel span/diameter. Rollers were applied to the two parallel planes within which the tunnel intersected (XZ-plane), as well as the plane that runs along the length of the tunnel (YZ-plane). Pins were applied to the three remaining external boundaries.

To validate the choice of model boundary size, the stress contours at the model boundary were measured prior to and after excavation. These tests were conducted at the highest insitu stress state (i.e. with a K = 3.5) in order to accommodate for the highest possible stress magnitude and contour depth/extent. One test, using a rectangular tunnel profile, was conducted in each of the three numerical modelling programs to ensure validity in each. It was determined that since the stress magnitude measured at the boundaries of each of the models remained unaffected after excavation of the tunnels (i.e. stress at model boundary in unexcavated model = stress at model boundary after excavation), the choice of model boundary size was indeed large enough. The affects boundary conditions were thereby negated by this appropriate choice of model size.
Mesh Size and Discretization

For the 2D RS\textsuperscript{2} numerical models, the discretization density of the mesh elements was increased around the excavation to employ a pseudo-radially gradational model mesh, which aids to better delineate the extent of the EDZ around the tunnel perimeter. The smallest mesh element, located along the excavation boundary, measured roughly 0.25 – 0.30 m in length. A 6-noded triangular mesh was used for each of the RS\textsuperscript{2} models presented in this thesis.

For the 3D RS\textsuperscript{3} numerical models, the discretization density of the mesh elements was again increased around the excavation to employ a pseudo-radially gradational model mesh. To strike a balance between model efficiency and optimum mesh discretization, a mesh boundary was used in the RS\textsuperscript{3} models. The size of this boundary measured half that of the total model boundary, thereby dividing the model into two evenly sized sections. A grading factor of 2 and an external grading factor of 0.5 was used for the entirety of the model. The mesh element density was then doubled within their interior portion of the model, generating mesh elements of 0.25 m in edge length along the excavation boundary (in an aim to be consisted with the RS\textsuperscript{2} models). A 4-noded tetrahedron mesh was used for each of the RS\textsuperscript{3} models presented in this thesis.

In order to validate the use and size of this mesh boundary, models with and without this mesh boundary were compared. Given the fact that both models yielded nearly identical results in terms of stress magnitude and contour extent, it was determined that the mesh boundary presented no negative implications to the modelling outputs. It is also worth noting that the mesh boundary lies well outside the plastic EDZ region and therefore did not impact the shape and extent of the yielded elements that define the EDZ/EIZ boundary.

For the 3D FLAC\textsuperscript{3D} numerical models, the discretization density of the mesh elements was again increased around the excavation to employ a pseudo-radially gradational model mesh. In order to maintain consistency with the RS\textsuperscript{3} models, mesh elements along the excavation boundary measured 0.25 m in edge length. Given that program’s nature and increased ease of
defining mesh gradation within the model code, a model mesh boundary was not used in the FLAC\textsuperscript{3D} models. Although the size and gradation of the mesh elements in the FLAC\textsuperscript{3D} models were nearly identical to those of the RS\textsuperscript{3} models, it should be noted that a cubed/rectangular-shaped mesh was used in FLAC\textsuperscript{3D}, as compared to the tetrahedral model mesh of RS\textsuperscript{3}.

To properly capture brittle damage development and propagation around the excavations, the mesh element size in each of the aforementioned programs was chosen in accordance with Walton and Diederichs (2015); the smallest mesh element measured no less than 3\% of the tunnel radius.

**Numerical Modelling Input Parameters**

**Cobourg Limestone Material and Strength Properties**

As previously outlined in Chapter 2 of this thesis, the material properties for the Cobourg Limestone serve as the inputs for each of the numerical models. Material and strength properties were obtained through testing conducted under contracts of the NWMO (NWMO 2015), including testing conducting by the Geomechanics Research Lab at Queen’s University. These tests include P- and S- wave velocity measurements, UCS testing (with acoustic emission monitoring), triaxial strength testing, Brazilian tensile strength testing, direct shear strength testing of the bedding planes, as well as long-term strength degradation testing. These tests were all carried out in accordance with the International Society of Rock Mechanics (ISRM) suggested testing methods (ISRM 2007, ISRM 2014). The threshold values for CI and CD were obtained from the AE monitoring conducted during the UCS testing of the samples.

Given the known material properties of the Cobourg Limestone, the DISL brittle damage model was used to reproduce the non-linear spalling behaviour model originally shown in Figure 2-3. This simplified constitutive model was developed by Diederichs (2007) and proven to accommodate a strain weakening behaviour constitutive model using either the generalized Hoek-
Brown model (Hoek et al. 2002) or an equivalent Mohr-Coulomb approach. Used with a non-linear FEM or FDM code, Diederichs (2007) explains that the DISL model can predict brittle rock spalling, as a function of confinement, within underground excavations. Equivalent, Mohr-Coulomb peak and residual envelopes can be determined to match the non-linear Hoek-Brown DISL envelope. The DISL parameters served as the input “peak” and “residual” material properties for the FEM and FDM numerical models presented in this thesis.

A strain limit value of $1e^{-3}$ was used to mobilize the cohesion and fractional strength components of the Cobourg Limestone material in the FLAC$^{3D}$ FDM models that utilize the CWFS brittle damage model approach. This value was chosen based on research published by Walton (2014). This value was obtained through a combination of laboratory testing and back analysis of excavations that were observed to experience brittle damage (Walton 2014).

Note that although the FEM models do not contain a strain dependency factor to mobilize the cohesion and fractional strength components (transition between peak and residual material properties in RS$^2$ and RS$^3$ is effectively instantaneous), the peak and residual input material properties/parameters in both the FEM and FDM models were identical.

**Stress Regime**

Given the density of the Cobourg Limestone unit, which is roughly 2,700 kg/m$^3$, as well as gravitational loading, Chapter 1 explains that the vertical stress within the DGR will likely lie between 13.2 MPa – 18.5 MPa (given an expected depth of 500 – 700 mbgs). Note that even though the subsurface at the proposed DGR sites present numerous stratigraphic layers with varying densities (NWMO 2015), this value was obtained by using an overburden of uniform/homogeneous unit weight for modelling simplicity. This vertical stress ($\sigma_V$) range was deemed valid given the fact that it corresponds to estimates and values from numerous data sources summarized and explained in Chapter 1 (Radakovic-Guzina et al 2015).
For each of the numerical models presented in this thesis, $\sigma_v$ serves as the minor principal stress ($\sigma_3$) since research summarized in Chapter 1 indicates that the horizontal stresses exceed the vertical stresses at the locations of the potential south-western Ontario DGR sites (Brown and Hoek 1978, Al et al. 2011).

Researchers (Adams and Bell 1991, Arjang 2001, Bauer et al. 2005, Radakovic-Guzina et al 2015) explain that the horizontal stress ratios lie between roughly 1.25 – 1.63 at the location of the three potential south-western Ontario sites. Given that fact that the specific APM DGR site has yet to be selected, a range of 1.5 – 3.5 was used for the purposes of this thesis in order to depict a suitable and conservative range within the overall Canadian context (as outlined by Brown and Hoek 1978, Al et al. 2011).

While the vertical stress serves as $\sigma_3$ in each of the numerical models presented in this thesis, $\sigma_1$ was input based on the selected major in-plane horizontal stress ratio. If $\sigma_2$ is not explicitly stated or listed in the introduction of Chapters 4 through 9, it was calculated based on an average of the vertical stress and the major in-plane horizontal stress (i.e. average of $\sigma_{v,3}$ and $\sigma_1$, respectively). This relationship can be seen in the various tables contained in Appendix A, which list the model stress regimes in terms of the major and minor in-plane horizontal stresses in relation to tunnel orientation.
References


NWMO. 2010. Moving forward together: process for selecting a site for Canada’s deep geological repository for used nuclear fuel. Toronto, Canada.


