TUNNELLING IN HORIZONTALLY LAMINATED GROUND: 
THE INFLUENCE OF LAMINATION THICKNESS ON
ANISOTROPIC BEHAVIOUR
AND PRACTICAL OBSERVATIONS FROM THE NIAGARA
TUNNEL PROJECT

by

Matthew Adrien Perras

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Abstract

The Niagara Tunnel Project is a 10.4 km long water diversion tunnel being excavated under the city of Niagara Falls, Ontario by a 14.4 m diameter tunnel boring machine. This tunnel has descended through the entire stratigraphy of the Niagara Escarpment, including dolomites, limestones, sandstones, shales and interbedded zones of these rock types, passed under St. Davids Buried Gorge ascending to surface. Working at the tunnel provided an opportunity to assess and document the horizontally laminated ground behaviour for this large diameter circular tunnel and provided the backdrop for this study. A detailed understanding of the geological history was necessary.

Modelling of laminations, ranging between 0.16 to 16 m in thickness, was conducted to determine critical behaviour and cut-offs for failure modes. A critical normalized lamination thickness (thickness/radius) of 0.9 was found to exist, above which the excavation response is similar to the equivalent isotropic model, and below which the laminated behaviour corresponds to a characteristic failure mode controlled by bed deflections and bed parallel shear. Initially, as the normalized lamination thickness is decreased below 0.9, the stresses are channeled through the crown beam which concentrates the yield and increases the crown deflections. This results in crown beam failure. As the lamination thickness decreases, further the stresses are shed to multiple laminations increasing the displacements significantly and changing the shape and extent of the yield zone. From multiple lamination coupling to self-limiting yield the development of chimney style failure is controlled by the degree of tensile yielding. Tensile yielding first begins in the haunch area and progressively extends above the crown, as the lamination thickness decreases, until a self-limiting plastic yield zone shape is reached at normalized lamination thicknesses below 0.026. Incorporation of discrete anisotropy is necessary to accurately model the excavation response in horizontally laminated ground.
Co-Authorship

The following thesis represents the original work of the author. Two conference papers, attached in Appendix B, were co-authored with Dr. Mark S. Diederichs.
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List of Symbols

C  Cohesion for failure of a plane or a rock mass property
D  Diameter of tunnel
Dh  Hydraulic diameter, a function of hydraulic radius
Ei  Young's modulus, measured from the elastic portion of the stress-strain curve
c  Roughness value, close relation to physical dimension of roughness projection
f  Darcy-Weisbach friction factor
GSI  Geological Strength Index for classification of rock masses
g  Gravitational acceleration
hf  Head loss due to wall friction
Q  Flow rate measured in cubic meters per second
Q  Rock Tunnelling Quality Index for classification of rock masses
Rh  Hydraulic radius, a ratio of cross sectional area to wetted perimeter
RMR  Rock Mass Rating system for classification of rock masses
RQD  Rock Quality Designation, for determination of intactness of rock masses
USC  Uniaxial Compressive Strength, a standardized laboratory test
V  Cross sectional mean flow velocity measured in meters per second
σn  Stress acting normal to a plane or surface
σ1  Major principal stress or maximum stress
σ2  Intermediate principal stress
σ3  Minor principal stress or minimum stress
τs  Shear stress acting on a plane or surface
ν  Poisson's ratio, a ratio of vertical to lateral strain
Chapter 1 : Introduction

1.1 Large Tunnel Boring Machine Challenges

Tunnel boring machines (TBM) have been used since the early 1850’s to create underground openings. Charles Wilson is credited with having developed the first successful continuous boring machine which was used on the Hoosac Tunnel in 1856 (Bickel & Kuesel, 1982). It was not until the early 1950’s however, that TBM technology became competitive against drill and blast techniques and James S. Robbins designed a TBM used on the Oahe Dam project in South Dakota. In the 70’s and 80’s TBM technology was beginning to be investigated in more detail, with the help of scientific organizations which started to conduct research into the physics behind mechanized mining. The technology has become a typical construction tool in tunnelling today and TBM’s have been used in a large spectrum of ground conditions. Large diameter tunnels present some unique challenges over smaller diameter tunnels, both in terms of equipment and stability challenges.

In larger diameter tunnels, there is more space for equipment, but much of this space is taken up by systems for maneuverability, such as gripper pads and cylinders, propel cylinders, conveyors and motors. These systems are essential to the operation of the TBM and leave less room for rock support equipment, such as drills, channel erectors and shotcrete robots. The cutterhead itself leaves a gap between the face and rock support installation area, where the rock is only supported by the cutterhead itself. This can allow the rock to dilate prior to installation of
rock support and in some instances, allows unsupportable deformations to occur, outside of the designed support capacity. Minimizing the gap between the face and rock support installation area is necessary (Steiner, 2000) to limit these deformations in lower strength formations. Rock support equipment should be flexible enough to install rock bolts at multiple angles, both vertically and horizontally. This maneuverability can be hindered by other systems. Automation of rock support equipment should be maximized due to heavy rock support components, which will speed production by increasing work productivity.

The stability of the tunnel is essential for the safe and effective operation of a TBM. Primary rock support is installed as close to the face as possible and secondary support can be installed further back. In large diameter tunnels, the effective span is greater and there is an increased potential for the influence of discontinuities to cause instability, such as wedges and unraveling.

Wedges can present a challenge in TBM driven tunnels both at the face and in the crown. Infrequent wedges at the face do not merit support installation, but they can cause problems with the disc cutters or conveyor system if they pass through the head. The scoops on the cutterhead are often fitted with “grizzly” bars, similar to mining ore passes, to prevent large blocks from entering the head. If frequent wedges or unraveling in the face will be a potential problem then face stabilization techniques must be made possible. This will require slots or holes in the cutterhead such that fiber glass reinforcement and or grouting can be installed or conducted. Wedges in the crown are less of an issue as long as they are observed and supported at the primary support installation location. TBM’s are often only fitted with two support installation locations, for open machines, and if a wedge is not supported at the primary location, then portable rock support equipment must be brought in, which will cause delays in production. In
horizontally laminated ground wedges can be common if vertical structure is present to complete the wedge geometry.

Both the maneuverability of the TBM, the location and maneuverability of the rock support equipment and the stability of the tunnel present challenges for large diameter tunnels. The design and operation of large diameter TBMs is a difficult task and careful consideration of the geological condition to be encountered will influence these decisions.

1.2 The Niagara Tunnel Project

The current Niagara Tunnel Project is being excavated by a 14.4m diameter Tunnel Boring Machine (TBM). The tunnel passes through the entire stratigraphy of the Niagara Escarpment, from the cap rock of the Lockport Formation down into the upper Queenston Formation. It passes under the St. Davids Buried Gorge to make its connection to the existing power canal system above the Sir Adam Beck Generating Station (SAB GS). This project will enable Canada to utilize more of its generating capacity from the Niagara River and help the owner, Ontario Power Generation (OPG) meet the growing demand for green power.

1.2.1 Power Generation from the Niagara River

The Niagara River flows in a south to north direction, from Lake Erie to Lake Ontario, with an average volume of 6000 m³/s (Harding, 2007). A 1950’s treaty between Canada and the United States of America controls the volume of water flowing over Niagara Falls to 2832 m³/s during the daytime from April to October and to 1416 m³/s at all other times (Delmar et al., 2006). The
remaining volume of water is shared equally between the two countries and when one is not utilizing the full capacity, the other can rent available power generating capacity (Harding, 2007).

Power generating on the Niagara River is now conducted through the SAB complex, which consists of SAB I and SAB II power stations. SAB I went into service by 1925 and SAB II by 1954. The SAB I was originally fed by the Chippawa canal system, which runs from the Welland River through the city of Niagara Falls. Along with the construction of the SAB II station, two 14 meter internal diameter tunnels were constructed, using drill and blast techniques, to divert the Niagara River water from just upstream of Dufferin Island, (see Figure 1.1). The water exits the twin tunnels, prior to crossing St. Davids Buried Gorge, into a canal system which joins the Chippawa canal system; see Figure 1.1 for arrangement. The canal system also connects to a pump storage generating station (PGS). The PGS can pump water into a reservoir during low demand periods and generate power, feeding the water back to the SAB GS, during peak demand.

With increasing demand for power in Canada, OPG has begun constructing the Niagara Tunnel, for water diversion, under the Canadian city of Niagara Falls, Ontario. The new tunnel will increase the 1800 m$^3$/s diversion capacity of the SAB complex to 2300 m$^3$/s, which will in turn mean that the available water for diversion to Canada will only exceed the SAB complex generating capacity 15% of the time, rather than the existing 65%; see Figure 1.2 below (Delmar et al., 2006).
Figure 1.1 Niagara River hydropower system layout (courtesy of Ontario Power Generation).
Figure 1.2 Water availability in the Niagara River showing exceedence of the SAB GS with the existing system (~65%) and with the addition of one tunnel (~15%) or two (~2%) (Delmar et al., 2006).

1.2.2 Alignment and Components

The Niagara Tunnel will divert water from the Niagara River, starting at the International Niagara Control Works (INCW), a structure which stretches out into the Niagara River to control flow for both the U.S. and Canadian diversion tunnels, see Figure 1.3. From the INCW, the tunnel descends down through the upper formations at 7.28% grade before leveling out at 0.1% downward grade once the tunnel is clear of the existing underground power infrastructure. The tunnel continues at a slight downward grade of 0.1% until it approaches St. Davids Buried Gorge where it descends at 7.28% to pass beneath the buried gorge. The tunnel levels out underneath the
gorge at an elevation of approximately 40m, or 140 m below ground surface, before ascending at 7.82% grade to exit into the Pump Generating Station (PGS) canal which connects into the Sir Adam Beck (SAB) canal system at the cross over location (Harding, 2007); see Figure 1.1 for plan and Figure 1.4 for long section. The alignment was modified from the original concept and design alignment after adverse tunnelling conditions in the Queenston Formation were encountered.

![Image of Niagara Falls]

Figure 1.3: Photo showing the tunnel intake location above Niagara Falls (photo courtesy of Ontario Power Generation).
Figure 1.4: The upper drawing is the original longitudinal section (modified from Perras & Diederichs, 2009) and the lower drawing is the new tunnel longitudinal section (as described in the text and from Everdell, 2009) of the Niagara Tunnel Project (original data courtesy of Ontario Power Generation).
Figure 1.5: Niagara stratigraphy visible from the Niagara River Gorge at the Whirlpool. The Whirlpool sandstone constrains the narrow outlet of the Whirlpool as shown.
1.2.3 Geological Overview

The tunnel will pass through 11 formations of the Appalachian basin, starting with the Lockport Formations and moving down through to the Queenston Formation, as shown in Figure 1.5. The Appalachian sedimentary basin, which covers most of southern Ontario, consists of Paleozoic strata deposited over Precambrian metamorphic rocks. It is bounded by the Algonquin and Findley arches, Precambrian basement highs in the West, the Frontenac arch in the North and the Taconic Mountain Range in the East and South, where the sediments were eroded during the Taconic and Appalachian orogens. As shown in Figure 1.6, the Algonquin and Findley arches are

Figure 1.6 Geological overview of Southern Ontario from Mazurek (2004)
a continuation of each other, separated by the Chatham Sag, a saddle point in the Precambrian basement rocks.

The Precambrian basement forms the foundation for the Appalachian basin. During the deposition of the sediments in the Appalachian basin, throughout the Ordovician and Silurian periods, the Findley and Algonquin arches are believed to have been actively subsiding because the sedimentary units thin over the arches (Stearn et al., 1979) and multiple unconformities reported by Mazurek (2004), after Cercone and Pollack (1991), and truncation of formations indicate intermittent up-lift of these arches during deposition. The Frontenac arch represents the topographic limit of the Appalachian basin.

The formations within the Appalachian basin lie relatively flat, dipping 6m/km (Yuen 1992) with local variations. The stress ratio (Ko) is reported to be in the order of 3-5 (Perras & Diederichs, 2007), with values ranging between 9 – 24 MPa in the Queenston Formation (Yuen et al., 1992). Generally speaking, the river systems tend to follow the regional joint systems and four joint sets were reported for the Niagara Tunnel Project. Topographically, Southern Ontario is relatively flat, with the exception of the Niagara Escarpment, a cuesta running from Lake Huron at the Bruce Peninsula to Rochester, New York. The escarpment has been cut into by river erosion at numerous locations and most notably by the Niagara River, which has created the world renowned Niagara Falls. Although the geology may appear relatively straight forward the Niagara Region has had construction challenges in the past and some key challenges face the Niagara Tunnel.


### 1.2.4 Construction Challenges

The Niagara Tunnel Project faces some key construction challenges which must be overcome to complete the tunnel and maintain operation. These challenges include high horizontal stresses, local variations in the stress field due to topographic extremes, large tunnel diameter to meet water volume requirements, 11 different rock formations, some units which swell, and a horizontal bedding structure.

Southern Ontario is known for its locked in high horizontal stresses, which can result in structural instability of underground excavations. Previous tunnels and cuts have experienced heaving inverts, wall closure and cracked concrete liners, such as the Hart Lake Tunnel (Lo & Yuen, 1981), the Toronto Power Station wheel pit (Bowen et al., 1976) and Thorold Tunnel (Bowen et al. 1976). These difficulties are related to the stress relief after opening of an excavation and represent a time dependant deformation process. Local variation in the stress field near the topographic extremes, such as the gorge and escarpment, play an important role in the stress levels, and stress shadows at lithological boundaries can create minor changes in magnitude.

The Niagara Tunnel is required to pass 500 m$^3$/s of water. To accommodate the internal water pressure, a nominal 0.6 m thick concrete liner will be installed. To pass the required flow and include the liner, the excavation diameter had to be 14.4 m. This creates a large span for the crown rock, and in combination with the horizontal bedding and high stresses, a difficult crown to support when utilizing a TBM for construction. This is because the cutterhead on the TBM, does not allow for rock support to be installed at the excavation face.

With 11 rock formations, many of which are less than 10 m thick, to be tunneled through, the variable ground conditions present a challenge both for excavation and for ground response.
With the unconfined compressive strength of the formation varying from 14 to 216 MPa (Perras & Diederichs, 2007), excavating the tunnel under mix faced conditions presents a challenge for the TBM operators and for the equipment. One formation will be easily excavated while the other is not and this means that the machine has different loads on different cutters at one instance. As the cutter moves from an easily excavated material to one more difficult, a ridge is created which has a sharp impact on the cutter as it passes by. This can increase cutter wear and also cause fractures in the cutters which result in increased cutter replacement and down time. The variable ground response from unit to unit can also present a challenge to rock support as one unit may be stable, but the overlying or underlying one is not. Under these circumstances the more conservative ground support should be installed to cover these transitions.

![Lithological boundary](image)

**Figure 1.7: Detection of lithological transitions for downward and upward TBM drives in horizontally laminated ground.**
The transitions are more accurately determined on a downward drive and can be difficult to determine on an upward drive as illustrated in Figure 1.7. This can present potential instability issues if the overlying unit is weak and the full weight of the unit must be supported as the stronger unit below thins.

The Queenston Formation and other argillaceous units in the Niagara Region have swelling potential. It has been found that the Queenston Formation can generate up to 4-5 MPa of swelling pressure (Rigbey and Hughes, 2007). The swelling process also generates fractures within the rock mass, which could promote further swelling at greater depths away from the excavation. Fresh water initiates the swelling process. The swelling issue has been mitigated for the Niagara Tunnel Project by the installation of a membrane to eliminate fresh water interaction with the Queenston (Rigbey & Hughes, 2007). Swelling will be discussed later and the reader is referred to Appendix A for further details on the swelling process.

The horizontally bedded nature of the sedimentary units of the Niagara Region creates anisotropic ground conditions. Wedges are easily developed with even a single vertical joint, as the bedding planes present the secondary detachment surface. Generally jointing is widely to very widely spaced in the Niagara Region and as such wedge failure at the Niagara Tunnel Project was not a continuous difficulty. The anisotropic ground conditions allow for increased lateral slip over an isotropic material which creates a different stress – strain response in the horizontal direction than the vertical direction. The plastic yielding due to overstressing of the rock mass is not predicted to the same degree using the traditional isotropic modeling approach. This is the fundamental problem that will be investigated in this research.
1.2.5 Big Becky – The Tunnel Boring Machine for Niagara

The TBM for the Niagara Tunnel Project is an open gripper, main beam machine with a diameter of 14.44 m. At the time of construction it was the largest hard rock TBM of its kind in the world. The TBM itself is approximately 45 m long with an attached 105 m long trailing gear system, including backup units 2-4 in Figure 1.8, which houses all the supporting components for the operation of the unit (Harding, 2007). There are 85 disk cutters on the head, each with a diameter of 508 mm (Harding, 2007). The cutter disks can be back loaded into the cutterhead to save time during replacement (Harding, 2007). In Figure 1.8, details of the TBM components are shown for reference.

The broken material is collected by scoops on the cutterhead, which is in turn fed to a conveyor belt at the center of the head. The cutterhead is powered by 15 electrical motors for a total of 6330 HP and 18 800 kNm torque at low speeds of 2.4 rpm (Harding, 2007). The recommended cutterhead thrust pressure is 28 MPa (Harding, 2007) and the grippers up to 21 MPa.

The cutterhead is 4.1 m long and an additional 2 m flexible finger shield, (Figure 1.8) extends from the back of the cutterhead. The finger shield is intended to allow for rock support installation under a protective cover. There are two rock drills located in the front area of the TBM, which are used to drill holes for rock bolt installation. The drills are mounted on a gear ring, centered about the main beam, which can travel forward and backward. The drills themselves can also rotate in the drilling plane to allow for multiple drilling angles. There is also a forward probe drill attached to a circular traveler allowing for movement both axially and radially. The forward probe drill, (Figure 1.9), is limited to a 30 degree inclination above the cutterhead and is used for advance probing, to determine if ground water seepage will be a
problem up ahead, for pipe spile installation, during the excavation near St. Davids Buried Gorge, and for scaling in the crown area above the cutterhead. A ‘donkey’ scissor lift brings C-channel and ring pieces forward for assembly and installation. Various working platforms were modified and replaced with two man lifts to increase the mobility of the support crews at the face.

The secondary support is installed almost 35 m from the working face, in the L2 area. Rock bolts, mesh and shortcrete can be applied at this location by track mounted equipment (Figure 1.9). The tracks allow for both radial and axial movement similar to the rock drills from the primary rock support location. Two shotcrete robots apply enough shotcrete to cover all rock bolts, channels, ribs and other support elements so that a smooth surface is achieved for installation of a water proof membrane. The water proof membrane is to be installed to eliminate the swelling of the shale rich rocks, namely the Queenston Formation. The swelling of the shale units will be discussed later.

The TBM uses grippers to hold the machine in place so that forward thrust cylinders can push the cutterhead forward for excavation. When the grippers are disengaged, legs are lowered to support the TBM (Figure 1.10) and allow the re-positioning of the grippers and walking legs.
Figure 1.8: A - photo of the TBM with trailing gear ready for launch, B – cutterhead with 85 cutter disks, C – a cutter disk, D – looking from back of cutterhead at scoop for muck removal and back loading chamber for cutter disk, and E – flexible finger shield as it enters the tunnel (photos courtesy of Ontario Power Generation).
Figure 1.9: A – L2 rock drill on circular track for movement, B – close up of rock drill, C – channels and mesh partially covered in shotcrete and D – full circular ribs being covered in shotcrete (photos courtesy of Ontario Power Generation).
Figure 1.10: A – TBM walking leg in position for re-grip, B – TBM walking legs in position for mining, C – gripper pad prior to launch, D and E – trailing gear entering the tunnel (photos courtesy of Ontario Power Generation).
1.3 Large TBM Excavation and Engineering Geology

Large diameter TBM’s have been used in a variety of geological conditions from clay soils under river channels (Li et al., 2008) to high stress environments under the Swiss Alps (Loew et al., 2000). TBM designs have improved to a point where mechanized tunneling methods are possible under the most extreme geological conditions. The difficulty now lies where the geological conditions bracket the extremes, such as thick fault zones in otherwise competent granite. Hybrid TBMs which take components from soft ground and hard rock machines are now being used.

TBM’s are generally of two types, soft ground or hard rock and both have been used to drive tunnels greater than 10 m in diameter. Currently the largest soft ground TBM is a slurry shield machine constructing a 15.43 m diameter tunnel under the Yangtze River for the Shanghai Yangtze River Crossing project (Li et al., 2008). The tunnel is 8km long and should be finished by mid 2010. The largest hard rock TBM is the one being used on the Niagara Tunnel Project, measuring 14.4 m in diameter, as discussed previously.

It is not uncommon to have large TBM excavations for infrastructure projects. The common highway in the United States must have lanes of 3.66 m width and vertical clearances of 4.27 m for minor routes and 4.88m for interstate highways (Bickel & Kuesel, 1982). The clearance for a single track rail tunnel in the United States is 4.88m width and 6.71m height (Bickel & Kuesel, 1982). Using these clearances, it is easy to see how multilane tunnels require large diameter excavations, greater than 8 m.

The geological conditions at a site play a pivotal role in determining the diameter of the tunnel and whether single, multiple, or stack lanes of traffic can be accommodated. Twin tubes, one for each direction of traffic, are often used for transportation tunnels for safety reasons. The
second tube can be used for evacuation in the event of an emergency. Careful consideration of the geological conditions must be taken into account when selecting the tunnel diameter. In weak rock, collapsed ground or ground water pressures may create overstressed conditions on the final lining. The diameter must accommodate the final lining thickness. A thorough site investigation program will determine the strength of the rock formations, the deformation properties, the stress levels, the ground water pressures and hydraulic conductivity. All of these will influence the design of the tunnel support and lining, which will in turn affect the tunnel diameter. For hydropower tunnels and over conveyance tunnels, the diameter is governed by the volume of water required and the geological conditions. Here there can be more flexibility in the diameter to meet the geological conditions and an optimum diameter with minimal frictional losses can be balanced against the adverse geological conditions if present. In either case, transportation or conveyance, the tunnel diameter will be optimized using numerical methods.

1.4 Numerical Methods

The simulation of rock using numerical methods is an essential tool for engineering and many volumes have been written on the subject (Brady & Brown 2006, Pande et al., 1990, Desai & Christian, 1977, to name a few). The underlying principle of any numerical method is to break down the problem into manageable components, which is called discretization. Either the governing equations can be discretized, as in finite difference methods, or the physical domain, as in finite element methods (Desai & Christian, 1977). Geotechnical materials exhibit non-linear behaviour, and analysis techniques use incremental or iterative approaches in solving the problem (Desai & Christian, 1977). Most numerical methods use displacements to calculate strains, which
then can be used to determine stresses using the material properties. Displacements and stresses are the primary values of interest to the engineer. A more detailed description of the Distinct Element Method and the Finite Element Method are presented in Chapter 4.

1.5 Thesis Objectives

Using the Niagara Tunnel Project as a back drop for the larger research focus, a detailed geological review of the Niagara Region was conducted and new hypothesis for the late geological evolution of the Niagara Region topography has been explored. The specific objective of this thesis was to determine at what thickness horizontal laminations become an important consideration for engineering design purposes, and how their explicit inclusion in a numerical model changes the yielding behaviour around circular excavations at shallow depths. These objectives have been achieved by:

1. Assessing and documenting the horizontally laminated ground behaviour at the Niagara Tunnel Project, the world largest hard rock TBM driven tunnel, including swelling potential of the shale units.

2. Identifying and understanding the failure modes and challenges of large diameter TBM driven tunnels.

3. Numerical simulation of the glacial activity was conducted to determine how the rock mass of the Niagara Region may have been affected by glacial activity, in order to refine the rock mass property assumptions
4. Assessing the rock mass yielding behaviour from the Niagara Tunnel Project and comparison to the numerical simulations, for verification of the failure mechanism.

5. Conducting numerous numerical simulations with varying lamination thicknesses between 0.16 m to 16 m.

6. Sensitivity analysis for stress, material and joint properties, and tunnel diameter were also conducted to constrain the limitations of model results.

7. Compilation of the model results to develop a chart showing the different anisotropic failure modes.

1.6 Problem Statement

The construction of large diameter tunnels in horizontally laminated rock presents some unique challenges which are not present while tunnelling with lamination at other orientations. Most road tunnels, water diversion tunnels and sewage tunnels are excavated with long sections that are less than 20 ° from the horizontal and therefore any horizontal features which present rock support challenges will be an ongoing problem during tunnelling.

Laminations can be of several forms; sedimentary bedding, a tectonic fabric, or horizontal joint sets. Each form, as well as each rock type, have different strength characteristics and so developing a generic empirical approach to rock support design under these conditions is difficult. Numerical simulation aids the designer; however the state of engineering practice does not adequately capture the mechanical behaviour of a horizontally laminated rock mass.
1.7 Summary of Findings

The summary of findings have been grouped into two sections consistent with the objectives of this research; i) the glacial impact on the rock mass of the Niagara Region, and ii) the ground behaviour of horizontally laminated rock masses.

1.7.1 The Glacial Impact on the Niagara Region

Through a study of glacial ice sheet dynamics and numerical modeling methods, the glacial impact on the Niagara Region was explored. The main topographic features which were in existence prior to the last glacial ice sheet advance in the Wisconsin age were the Niagara escarpment and St. Davids Buried Gorge. These two features, created indirectly or directly by glacial activity, allowed for more than surficial rock mass yielding to occur.

The numerical modeling results, reported in section 2.3.7.1, indicate that tensile fracturing, either induced through direct loading or hydrofracturing could occur 30 m below the toe of the escarpment and 75 m back from the face, under wet conditions. Tensile failure in the stiffer units, such as the Irondequoit limestone and Whirlpool, sandstone can extend for 100’s of meters back from the face under wet conditions. This damage at the Niagara escarpment is dramatically increased when a gap is included on the highland of the escarpment. Intense fracturing from glacial activity may be over printed or removed by continued deformation of the escarpment face, post ice sheet retreat, although some of the jointing in the stiffer units of the Niagara region well back from the escarpment may well be due to glacial impact at the escarpment.
The formation of St. Davids Buried Gorge was reviewed and a new theory was explored. The new theory suggests that the gorge may have been formed by high pressure water underneath an ice sheet, creating a tunnel valley. Direct loading of an ice sheet on an underlying open valley below creates similar damage around the valley as river erosion with high horizontal stresses. With the addition of 100 m of head pressure above the valley lip, tensile failure is induced in the Rochester and Neagha formations several hundred meters back from the valley wall. Tensile failure is limited to near the valley surface when river erosion is modeled and the primary failure mode is in shear. Tensile failure becomes catastrophic throughout the model when greater than 100 m of head is modeled. Water pressures greater than 100 m could easily be achieved under 500 – 1300 m of ice cover and may have been relieved by tunnel valley processes, which would reduce the regional impact and localize the rock mass damage to near the tunnel valley. The sheared planes observed at the base of St. Davids Buried Gorge in the tunnel would have been created during or post gorge erosion as a stress relief mechanism whether the gorge was formed by tunnel valley process or river erosion. Certainly the gorge would have been used for river erosion at any event after it initial formation.

The rock mass has been locally affected by the development and existence of the topographic extremes in the Niagara Region. Consideration of the geological history is important to gaining a better understanding of the rock mass response to a large diameter excavation.

1.7.2 Failure Modes of Anisotropic Ground around Circular Tunnels

The state of engineering practice is to treat the rock mass as a homogeneous isotropic material. Using a classification system, such as GSI (Hock et al., 2002), heterogeneities in the rock mass
are accounted for and the rock mass strength parameters are reduced accordingly. It was found that explicitly including laminations within the rock mass, using joint elements, resulted in higher vertical crown deflections than the equivalent isotropic material. Elastic transversely isotropic model results were the same as an explicitly modeled elastic anisotropic model, which can be used for stress concentration analysis; however the displacements of the elastic model do not include the component resulting from plastic yielding. The anisotropic model with explicit laminations was found to be similar to the isotropic model results above a lamination thickness of 7250 mm. Below this cut off, the isotropic model inadequately represents the mechanical behaviour of the laminated material, because the horizontal slip is not accounted for, and it is suggested that anisotropic analysis be conducted.

The anisotropic analysis indicates that four modes of failure can be anticipated for unsupported circular tunnels in horizontally laminated ground, which includes gravity driven unraveling, localized haunch instability, crown beam failure and chimney failure. Gravity drive unraveling failure was delineated using voussoir analysis methods and is included for completeness. Localized haunch instability occurs in the quadrant above the spring line and below the crown elevation. Failure and fall will occur where localized structure interacts with the laminations and the tunnel periphery. Crown beam failure is caused by stress flow concentrated through the crown beam, which over stresses the beam and causes plastic yielding. This zone is also accompanied by localized haunch instability. Within the chimney failure zone, an unsupported tunnel will collapse in a near vertical fashion above the crown of the tunnel. The void can be characterized by steep sides dipping away from the excavation, in section. The height of the void and the vertical crown deflections will stabilize below a lamination thickness of
280 mm. These modes of failure can be used to predict tunnel performance prior to support installation, so that the support design process can take into account the failure mechanism.

1.8 Thesis Outline

This thesis is structured as a traditional thesis. Each chapter has an introductory section which provides the relevant information from literature and is followed by more detailed discussion of the research.

Chapter Two consists of a literature review of tunneling in horizontally laminated ground using Tunnel Boring Machine (TBM) excavation methods and a review and extrapolation for modeling purposes of the geological influences on the Niagara Region. Conventional support design and TBM excavation methods are reviewed as a general background for completeness. The geological engineering challenges of horizontally laminated ground are reviewed in more detailed and are followed by brief case histories. The Niagara Tunnel Project, a 14.4 m diameter tunnel in the sedimentary rocks of the Appalachian basin, is the main case history and is reviewed in more detail from early hydropower developments in the Niagara Region to the geological setting and new hypothesis for glacial impact on the topographic features of the Niagara Region are explored briefly. The author’s experience from the Niagara Tunnel Project provides the back drop for the larger focus of this research, tunneling in horizontally laminated ground.

Chapter Three is a discussion on classification using various systems available to the geological engineer. The properties of the Niagara stratigraphy, the rock mass units exposed along the Niagara escarpment, are reviewed and experience from field work on the Niagara Tunnel Project is used to correlate between classification systems and the observed behaviour.
The various failure modes, from swelling to stress induced failure, which were observed, are discussed in detail.

Chapter Four is a review of numerical methods which can be used for modeling horizontally laminated ground, and preliminary model results are used as examples to illustrate the strengths and weaknesses of each numerical method used in this research. It is an introduction to the work presented in Chapter Five for developing a preliminary, numerically based, failure mode chart which can be used to assess the potential extent of excavation damage around a circular excavation at shallow depth in horizontally laminated ground.

Chapter Five presents the numerical modeling techniques and validation of the methods used to develop the failure mode chart and discusses in more detail the various rock mass behaviour modes of horizontally laminated ground.

Chapter Six provides discussion on excavation design recommendation based on the behaviour modes as identified in the field and through the numerical modeling results. This discussion is intended to be an extrapolation of the results from Chapter Five. A rigorous support design analysis for horizontally laminated ground is beyond the scope of this research and is not presented here in detail.

A summary and discussion of the main findings of this research are presented in Chapter Seven. The limitations of this study are identified and future work is suggested as this research is applicable to nuclear waste storage in sedimentary rocks, such as the Georgian Bay or Queenston Formation at the Bruce site for Canadian nuclear waste.
Chapter 2 : Tunnel Construction and Geological History of the
Niagara Region¹

2.1 Tunnel Construction

Tunnel construction has been ongoing throughout the world since recorded history began (Bickel & Kuesel, 1982). Drill and Blast and TBM construction methods are both practiced today and there are many different types of TBMs utilized. In either case, there are significant engineering challenges for tunnels transecting horizontally laminated sections.

2.1.1 Conventional Support Design

The design of rock support is critical for the safe and efficient excavation and operation of any tunnel. The engineer will design rock support types which should cover all of the expected rock conditions anticipated to be encountered during excavation. Support utilizes and conserves the inherent strength of the rock mass, such that it becomes self supporting (Brady and Brown, 2006). Rock support in tunneling is often referred to as primary or secondary. Primary support refers to the support installed immediately after excavation. This ensures a safe working environment for other tunnel operations and final liner, secondary support, installation. The secondary support enhances the primary support, such that the tunnel will remain serviceable for the design life.

¹ Presented as part of a conference paper by Perras & Diederichs (2007) and included in Appendix B.1.
Primary support can be active or passive in nature. Active support applies a load to the excavation surface using tensioned rock bolts or cables, expandable concrete segments or powered support such as props. It is primarily used to support gravity driven loads (Brady & Brown, 2006). Passive support gains its loads as the rock mass deforms and can be provided by steel sets, timber blocking or sets, or un-tensioned rock bolts, such as grouted bars or other frictional bolts (Brady & Brown, 2006).

Secondary support can also be active or passive. In pressure tunnels voids behind the final liner must be filled with grout so that the internal water pressure does not damage the liner. If the grouting process is conducted using pressure, then the final liner and the rock mass will be married together and the final liner will exert a force on the rock mass. This is also achieved during pre-cast segment lined tunnels where grouting is necessary. The final liner for road and rail tunnels and other infrastructure tunnels, where the internal pressure is low, may have passive secondary support for aesthetic and safety reasons. Support design must take into account the interaction between the rock mass and the support (Hock & Brown, 1982).

The rationale behind design of support for underground openings is to minimize the deformations which are allowed to occur between the excavation face and the support installation area. Some deformation is required, such that the support pressures required to stabilize the excavation are manageable, however, the deformation must be limited such that the rock mass does not loosen, reducing its capacity to carry the loads (Brady & Brown, 2006). A ground reaction curve is prepared to determine where the optimum distance from the face occurs for support installation so that the support does not become overloaded.
This process is often called the convergence-confinement method and recent work by Vlachopoulos and Diederichs (2009) have improved and simplified its application for deep tunnels. An example of the convergence-confinement method is illustrated in Figure 2.1. The convergence-confinement method is a preliminary approach to support design, and baseline support types can be developed using this method. A ground reaction curve analysis should be conducted for each rock condition to be encountered. Hoek and Brown (1982) give detailed instructions on this process, with worked examples.
Figure 2.2. Wedges in the haunch area of a circular, arch-shaped crown developed along horizontal bedding planes and vertical jointing. Example taken from Pells (2002) for the Hawkesbury Sandstone in Australia.

The convergence-confinement method is a good approach for preliminary support design and allowance for additional local support must also be considered for gravity and sliding wedges. Gravity driven wedges can easily develop in a horizontally laminated rock masses, as illustrated in Figure 2.2, as sub-vertical joints can interact with any number of horizontal laminations to create a wedge in the haunch area. Reinforcement calculations can be done using limit equilibrium, if the stresses are ignored. Limit equilibrium can also be used for sliding wedges and the resulting support requirements are conservative when the stresses are ignored (Brady & Brown, 2006).

For more complex geometries and failure processes, comprehensive quantitative solutions are only available for preliminary support design (Brady & Brown, 2006). More complex analytical and semi-analytical solutions are available and Brady and Brown (2006)
suggest the following authors; Anagnostou and Kovari (1993), Brown et al. (1983), Carranza-Torres et al. (2002), Carranza-Torres and Fairhurst (1997) and (1999), Detourney and Fairhurst (1987), and Wang (1996). For final design purposes, numerical methods are used to verify assumptions and carry out further sensitivity analysis.

Empirical design rules from Lang (1961) were developed in the early stages of rock mechanic studies (Brown, 1999) and further developments in the mid 1970’s by Bieniawski (1973, 1976) and Barton et al. (1974) used practical experience and precedence from projects to develop classification schemes, such as RMR and Q, respectively. Using these schemes, as discussed earlier, rock support estimates can be made, however Hoek and Brown (1982) clearly warn that these rock support recommendation should be used carefully, especially when the rock mass properties and excavation geometry differs from those used to develop them.

Hoek and Brown (1997) developed the Geological Strength Index (GSI) such that rock mass properties could be derived for direct input into numerical models, using the GSI value and laboratory derived values of m_i, E_i, and UCS. The GSI system is widely used around the world for numerical based deformation and rock support design analysis.

Inferences from the yielding process in horizontally laminated ground, presented in section 4 and practical observations from TBM driven tunnels have led to a number of practical suggestions for rock support implementation in horizontally laminated ground. It should be noted that the research conducted focused on the failure process and these suggestions require further analysis to determine their implementation in rock support schemes.
Table 2.1: Hard Rock versus Soft Ground TBM characteristics, based on Beckel and Kuesel (1982).

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Hard Rock</th>
<th>Soft Ground</th>
</tr>
</thead>
<tbody>
<tr>
<td>Excavation Material</td>
<td>Soft Rock – Shale, tuff, claystone, siltstone, sandstone</td>
<td>Firm – stiff clays and cemented or cohesive granular</td>
</tr>
<tr>
<td></td>
<td>Medium Hard Rock – Some basalt, granite, and andesite; average sandstone and limestone; dolomite, chalk, rhyolite, gneiss, and schist</td>
<td>Raveling – slightly adhesive sands and silts with apparent cohesion</td>
</tr>
<tr>
<td></td>
<td>Hard Rock – Some basalt, granite and andesite; well cemented sandstone and limestone; marble, chert, diorite, quartzite, argillite</td>
<td>Running – dry sand and clean, loose gravel</td>
</tr>
<tr>
<td></td>
<td>Hard Rock – Some basalt, granite and andesite; well cemented sandstone and limestone; marble, chert, diorite, quartzite, argillite</td>
<td>Flowing – saturated raveling or running ground where seepage may occur</td>
</tr>
<tr>
<td></td>
<td>Flowing – saturated raveling or running ground where seepage may occur</td>
<td>Squeezing – soft to medium plastic clays</td>
</tr>
<tr>
<td>Cutting Tool</td>
<td>Disk /Roller</td>
<td>Drag</td>
</tr>
<tr>
<td>Cutter Layout</td>
<td>Center - Roller cutters to ensure face stability</td>
<td>Same drag cutter over entire face</td>
</tr>
<tr>
<td></td>
<td>Face – main cutting area</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Gauge – controls diameter</td>
<td></td>
</tr>
<tr>
<td>Shield</td>
<td>Open (no shield) – used where standup time is significant</td>
<td>Fully shielded – Required to hold up soft ground material such that a liner, typically pre-cast segments, can be erected within the shield</td>
</tr>
<tr>
<td></td>
<td>Crown Shield – used to protect workers from wedges and loosened material</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Full - used where continuous overbreak may occur or water inflow is a problem</td>
<td></td>
</tr>
<tr>
<td>Support</td>
<td>Delayed – at a minimum the length of the cutterhead, typically 3-6m</td>
<td>Immediate – shield acts as temporary support until liner is erected</td>
</tr>
</tbody>
</table>
2.1.2 Tunnel Boring Machine Excavation

TBM’s are generally used for long tunnels due to the substantial upfront cost of the equipment. It is more cost effective to use drill and blast techniques for shorter tunnels because the equipment costs are much lower. The advantage of a TBM comes into effect when the tunnel alignment is long and continuous and the project schedule is long enough to merit the upfront cost of the TBM. There are many types and applications of TBM’s, but they generally fall into two main categories; Hard Rock and Soft Ground machines. Table 2.1 highlights some of the major differences. Beckel and Kuesel (1982) refer to hard rock as firm and cohesive material and soft ground as soft, plastic and non-cohesive material. It should be noted that there are hybrid machines which cross the boundaries between the two main types and that each TBM is designed specifically for a certain project. It is rare that a TBM is re-used for a different project in the same form as the original project; however, older TBM’s can be retrofitted or major components adapted to new projects. Since this research is focused on hard rock tunneling only, the hard rock TBM’s will be discussed in further detail.

2.1.2.1 Hard Rock Tunnel Boring Machines

Hard rock TBM’s are used in most rock formations where the material would otherwise require blasting for excavation. There are three main types of hard rock machines and the selection for a project depends on the ground conditions expected. The three types are, open (no shield), single shield and double shield and their main advantages and disadvantages have been highlighted in Table 2.2 after Shahriar (2007).
Table 2.2: A comparison of hard rock TBM types from Shahriar (2007)

<table>
<thead>
<tr>
<th></th>
<th>Open TBM / Crown Shield</th>
<th>Single Shield TBM</th>
<th>Double Shield TBM</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Advantages</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Easy operation</td>
<td>Easy operation</td>
<td>Wide range of application</td>
<td>Wide range of application</td>
</tr>
<tr>
<td>Applicability in hard rock</td>
<td>Applicability in hard rock</td>
<td>Safety</td>
<td>Safety</td>
</tr>
<tr>
<td>Support system flexibility</td>
<td>Support system flexibility</td>
<td>Precast segmental lining</td>
<td>Support system flexibility</td>
</tr>
<tr>
<td>High excavation rate</td>
<td>High excavation rate</td>
<td>High performance</td>
<td></td>
</tr>
<tr>
<td>Less construction cost</td>
<td>Less construction cost</td>
<td>Working in falling ground</td>
<td>Working in falling ground</td>
</tr>
<tr>
<td>Low investment cost</td>
<td>Low investment cost</td>
<td></td>
<td>Controlling water inflow with closed shield</td>
</tr>
<tr>
<td><strong>Disadvantages</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Grippers inability in unstable rock mass</td>
<td>Grippers inability in unstable rock mass</td>
<td>Two work phases</td>
<td>High investment cost</td>
</tr>
<tr>
<td>Support installation in weak rock masses</td>
<td>Support installation in weak rock masses</td>
<td>Complex operation</td>
<td>Complex operation</td>
</tr>
<tr>
<td>Need of precast lining</td>
<td>Need of precast lining</td>
<td>Need of cleaning the telescopic joint</td>
<td></td>
</tr>
<tr>
<td>Drive in weak ground</td>
<td>Drive in weak ground</td>
<td>Possibility of TBM jamming in highly convergent ground</td>
<td></td>
</tr>
<tr>
<td>Need of segment plant near by</td>
<td>Need of segment plant near by</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Hard rock machines use cutter disks to break the rock through tensile splitting. The cutter disks are typically spaced approximately 8 cm apart, such that concentric rings are formed on the excavation face and tensile fractures interact between each ring to produce chips (Villeneuve, 2008). The chips can range in size depending on the spacing of the cutters and the penetration rate, which affects the tensile fracture growth, creating the chips. The chips are
collected at the invert with scoops on the cutterhead and fed onto a conveyor or train system for final removal from the tunnel, as discussed earlier.

In non-shield machines, rock support can be installed behind the cutterhead or when required through the cutterhead to provide face reinforcement. Support installation for TBM driven tunnels is similar to other underground excavations, where rock bolts, wire mesh, steel sets and shotcrete can all be applied as part of the support system. There are space limitations with the installation of primary support from the TBM and long rock bolts or cable bolts are not necessary practical. As well, shotcrete application can block up mechanical tracks for equipment and interfere with electronics for TBM and support operations. These limitations can be minimized by designing the TBM around the rock support system, rather than around the maneuverability systems.

All TBM’s advance in a similar manner, using thrust cylinders to propel forward either off grippers which are locked into position against the rock for open machines, or the last support ring installed for shielded machines. The machine thrusts forward, dragging the trailing gear with it as the machine excavates. Once a thrust cycle is completed, typically 1-2 m of advance for large diameter TBMs, the thrust cylinders are reset and the cycle repeats. For open gripper and double shielded machines, rock support installation can occur during the mining cycle, although for heavy support applications the mining cycle may be too short a duration to complete the support installation in that time frame. With single shielded machines, advance must be completed prior to erection of support and typically the support comprises of pre-cast segments which are installed at a greater distance from the tunnel face than support from open type machines. With all TBM styles, the rock mass is unsupported for a minimum of several meters
prior to installation of support and this distance is often greater for shielded machines. This is perhaps one of the greater challenges faced with TBM driven tunnels.

2.1.3 Typical Engineering Geology Challenges with Tunnel Boring Machines

Each TBM is designed for the optimum site specific conditions and with more challenging tunnelling projects being constructed; the use of hybrid machines is becoming more popular. These challenging projects often have geological conditions which span the extremes in rock support requirements, from unraveling fault zones to rock bursting brittle failure.

2.1.3.1 Sidewall Instability

Grippers are used on open TBM’s for advance by pushing into the sidewall to create a point from which to thrust the cutterhead forward. The grippers must make contact with the sidewall over a large portion of the gripper face in order to utilize maximum thrust for advancement. In high horizontal stress situations, sidewall damage can occur through tensile yielding. If the depth of yielding is not uniform along the sidewall, then the grippers may not be able to make sufficient contact for advance thrusting. In this situation, shotcrete, concrete or timber/steel cribbing must be used to create a pad for the gripper to rest on, which causes delays in the production cycle.

2.1.3.2 Pre-Support Dilation

The point at which primary rock support can be installed is governed by the length of the cutterhead and the associated drive motors. In some machines, this distance can be in the order of 3-6m and results in delayed rock support installation. If the rock mass is to deform plastically or
there are a large number of joints to form wedges, the distance between the face and the primary rock support installation point should be minimized, so as to limit the amount of dilation of the rock mass. Once the rock begins to dilate, then it is exceedingly difficult to reinforce the tunnel without leaving loose blocks hanging from the support or removing the loosened and dilated material. Under extreme conditions, the TBM head can become jammed due to plastic yielding of the rock mass, where dilation occurs to such an extent that the rock lies on the cutterhead over much of its surface. In this situation the friction between the head and the rock can become too great and the head may be unable to be rotated due to the weight of the material. To minimize the effect of dilation onto the cutterhead forward spiles can be installed, which involves the placement of metal poles in holes drilled ahead of the face to create an umbrella which the machine can pass under. Spile installation operations can be time consuming and expensive.

Even in pre-cast segmentally lined tunnels, the length of the shield can contribute to uncontrolled rock mass dilation and fall out prior to the expansion of the lining ring and grouting of the annulus gap. At the Lesotho Highland Water Project, rock falling onto the shield caused eccentric loading of the liner during advance thrusting, which caused cracks to develop in the pre-cast segments (Graff & Bell, 1997). Extensive grouting behind the liner was required to prevent liner cracking due to the internal water pressures.

Minimizing the gap between the face and the installation of support on all TBM types can have benefits of early placement of primary or secondary support, which in turn will reduce rock mass dilation effects and potentially rock support requirements (Steiner, 2000).
2.1.3.3 Roof and Side Wall Wedges

Continuous observation by trained personnel can identify roof and side wall wedges as they are exposed at the back of the cutterhead. The early identification of possible wedges is necessary for TBM driven tunnels due to the limited position of rock support installation equipment. It is important to make an estimate of the size of the wedge as early as possible, and this can be done on the same interval of rock support installation (i.e. if support is installed over 1.5 m, then estimate the maximum probable wedge size and weight and install appropriate support at that location).

The difficulty arises if a wedge is not identified at the primary rock support installation location or the wedge geometry is such that the detachment or sliding plane is roughly perpendicular to the tunnel axis. If the wedge is not identified and supported at the primary support location, then portable rock drills may have to be brought in, since the secondary support location is often tens of meters back from the primary location. If the geometry of the wedge is such that it can move prior to the installation of the primary rock support, then tunneling will have to proceed with extreme caution. An example of a roof wedge, near the Whirlpool – Queenston contact from the Niagara Tunnel Project, is shown in Figure 2.3.
Figure 2.3: Photo of a roof wedge at the Niagara Tunnel Project. Upper photo was taken before collapse, with cracking obvious on joint surface and undermining near the base. The lower photo was taken after collapse, showing the block hung up on drilling equipment. Wedge detachment in front of the rock support installation area (photos courtesy of Ontario Power Generation).
2.1.3.4  Faults and Face Instability

Faults present a challenge for TBM excavations because they can contain high water pressures and soil-like material. This combination can cause costly delays, such as those on the Gotthard base tunnel when the Gobi II machine encountered a fault (Moergeli, 2007). The ground up rock fragments from the fault inundated the cutterhead due to the high water pressures and jammed the cutterhead. Spiling in combination with grouting has been used as an effective means to tunnel through faults by preventing unraveling of the ground above the TBM.

Face instability can be caused by jointing or weak rock. Face wedges caused by jointing can cause larger than desired rock fragments to be scooped up by the cutterhead as they are not effectively broken down by the disk cutters. These larger fragments can cause excessive conveyor belt wear and may damage other parts of the mucking system. Typically nothing is done for intermittent face wedges, however for consistent wedge problems some face support could be used, such as fiber glass rods. Fiber glass rods have been installed under weak ground conditions in order to advance the tunnel. The rods are easily excavated by the TBM. Other face conditioning, such as grouting, can be conducted through the cutterhead to provide some cohesion to the weak ground, long enough to advance the tunnel one mining cycle.

Tunnelling through faults and installing face support is costly and time consuming, reducing advance rates. Many successful projects from around the world have been completed under such conditions. Other geological challenges have also been met, as mentioned above, and have been successfully overcome. Geological observations at the tunnel face should be routinely conducted to monitor the ground conditions and identify problem areas which may arise.
2.1.4 Geological Engineering Challenges of Horizontally Laminated Ground

Tunneling in horizontally laminated ground presents some unique challenges for the geological engineer. Some of these challenges are listed below;

- Plane of weakness follows typical tunnel alignments.
- Regional stress variations can occur at major lithological boundaries.
- Non-horizontal structure can easily form wedges, using the excavation surface and horizontal plane of weakness for detachment.
- Perched ground water due to restricted vertical flow.
- Anisotropic strength and stress response.

These challenges can be met for both drill and blast and TBM driven excavations, as long as the engineer is aware of the issues. The precise method of dealing with these challenges will be different depending on the excavation method utilized; however the principles will remain consistent.

For TBM driven tunnels, the grade of the excavation is typically shallow. The approximate grade limit of rubber wheeled vehicles in tunneling applications is 8% and less for rail traffic. Grades of greater than 8% require cogged equipment to transport supplies, personnel and muck. Minimum grades should be 0.5-1.0 % to facilitate drainage (Bickel & Kuesel, 1982). This means at shallow grades that horizontal laminations will follow the longitudinal profile of the tunnel for long distances. This provides a plane of weakness in the crown which can be easily exploited by inclined joints to form wedges. This also creates a beam of rock in the crown which can deflect and become unstable. Such beams are described as voussoir beams and analysis
procedures for determining their stability have been described by Diederichs and Kaiser (1999) using an iterative calculation process.

Stress shadows can be created where units of largely different modulus overlie each other. The stiffer unit will attract the load of the stress field and the softer unit will have less stress flowing through it next to the contact for several meters. The lower confining stresses reduce the potential formation of a natural arch. Without the development of the natural arch, blocky or weak rock masses will collapse due to gravity raveling.

In horizontally laminated ground, perched ground water tables can be encountered, since flow may be restricted in the vertical direction in a particular unit. For instance, a karstic limestone layer could lie between two shale layers which act as aquitards. In such cases pre-grouting should be conducted to isolate the rock mass surrounding the tunnel to prevent water ingress, especially if the perched aquifer is connected to a local recharge source such as a river or lake.

Brittle failure is traditionally thought to occur in stiff rocks, such as granites. Brady and Brown (2006) define brittle fracture as the sudden loss of strength with little to no permanent (plastic) deformation. It is associated with strain-softening. Typically this occurs due to the concentration of stresses around micro-defects in the rock, which results in fractures propagating from this location when the stresses are high enough. The fractures are able to develop without interacting with each other, however, at some stage crack coalescences occurs and the rock strength decreases rapidly. When this failure process occurs in stiff materials under high stress it can result in rock bursts, violent explosions of rock. Generally speaking, the process of brittle fracture, whether violent or not, can result in spalling failure. In laminated rock masses, the damage on the excavation boundary perpendicular to the lamination plane will generally be less
severe as there the rock mass will be able to slip along the laminations more easily before brittle failure occurs. However, this process of brittle failure is highly dependent on the strength of the rock mass and the stresses.

The objective of this research is to understand anisotropic strength and stress response in horizontally laminated ground. Under such conditions, the traditional approach of using isotropic material properties is ineffective at determining the extent of plastic yielding. Explicitly including horizontal laminations will give rise to the proper mechanical behaviour of the rock mass.

2.1.5 Tunnels in Horizontally Laminated Ground

Numerous tunneling projects from around the world have encountered horizontally laminated ground. Some of the notable projects are reviewed briefly below for comparison with the Niagara Tunnel Project. These include the Donkin-Morien tunnels in Nova Scotia, Canada, the Navajo Water Diversion Tunnels in Nevada, USA, and the Gotthard base tunnel, Switzerland.

2.1.5.1 Donkin-Morien Tunnels

The Donkin-Morien tunnels provide access to the Harbour coal seam which is located off the coast of the Donkin-Morien peninsula in Nova Scotia (Marsh et al., 1986). The tunnels are 7.6 m in diameter and were driven by TBM, the Lovat M-300, which was the first Canadian rock TBM (Seedsman, 2009). The tunnels reached depths of 180 m below sea level, staying below the McRury Seam in a thick sandstone unit and are 3.58 km long (Seedsman, 2009).
Table 2.3: Strength Parameters for the rock types of the Donkin-Morien tunnels after Seedsman, 2009.

<table>
<thead>
<tr>
<th>Type</th>
<th>Lithology</th>
<th>1987</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>UCS (MPa)</td>
</tr>
<tr>
<td>I</td>
<td>Sandstone</td>
<td>92</td>
</tr>
<tr>
<td>II</td>
<td>Interbedded sandstone and siltstone</td>
<td>121</td>
</tr>
<tr>
<td>III</td>
<td>Siltstone</td>
<td>53</td>
</tr>
<tr>
<td>IV</td>
<td>Interbedded siltstone and mudstone</td>
<td>36</td>
</tr>
<tr>
<td>V</td>
<td>Mudstone</td>
<td>36</td>
</tr>
<tr>
<td>VI</td>
<td>Carbonaceous mudstone</td>
<td>16.6</td>
</tr>
<tr>
<td>VIA</td>
<td>Coal</td>
<td>16.6</td>
</tr>
</tbody>
</table>

The geology of the tunnel length was broken down into seven rock types as outlined in Table 2.3 and the mudstone, type VI, over a coal seam (type VIA) was deemed the worst loading condition, as reported by Seedsman (2009).

The tunnels were allowed to flood in 1992 after the coal project was abandoned and in 2007, after the Province of Nova Scotia issued a call for development of the coal resources, the tunnels were reopened (Seedsman, 2009). After dewatering, failures in the Donkin-Morien tunnels were observed to be limited to areas where minor coal seems were intersected, such as at the Emery and Bouthillier seams. These roof collapses were chimney shaped as shown in Figure 2.4. In other sections of the tunnel in weaker strata, side wall damage has occurred, and rocks have fallen out above the spring line.
Seedsman’s work (2009) showed that the brittle failure criterion with transverse anisotropy and low spalling limits can be used to determine the height of failure for the rocks at the Donkin-Morien tunnels. Without the transverse anisotropy, the tangential stress in the crown will be under predicted and the extent of brittle failure will be inaccurate (Seedsman, 2009). Figure 2.5 compares the failure analysis using isotropic elasticity with transverse anisotropic elasticity. This case shows the need to include anisotropy in numerical studies for underground excavations at shallow depth.
2.1.5.2 Navajo Tunnel No. 3

The Navajo Water Diversion Tunnels are part of an irrigation system in New Mexico, which takes water from the San Juan river (Sperry & Heuer, 1972). Sperry and Heuer reported on the construction of the tunnel at the 1972 Rapid Excavation and Tunnelling Conference and the following information comes from their paper. The Navajo Tunnel No. 3 is one of six tunnels in the system and is 4.65 km long with an excavation diameter of 6.25 m. The tunnel was constructed by TBM, which was launched from a blasted chamber. The tunnel was constructed in the asymmetric San Juan Basin, which contains sedimentary rocks dating from Cambrian time to recent. The tunnel was constructed in the Paleocene Nacimiento formation which is a silty to slight silty sandstone containing trace lime. The formation also contains shale lenses which are
randomly distributed and sized. The shale lenses are poorly compacted and flaky to blocky and
the siltstone is moderately friable to soft. On average, the materials had compressive strengths of
10 MPa, with the sandstone ranging from 2 - 67 MPa, the siltstone from 4 – 45 MPa and the shale
as low as 1 MPa. Three failure modes are reported by Sperry and Heuer (1972) for the Navajo
Tunnel No. 3, which are;

1. Failure in massive, homogeneous, dry material.

2. Failure in dry material associated with discontinuities, partings, lenses or low strength
   beds.

3. Failure associated with shale in the presence of water.

In failure class 1, fractures in the arch developed between the cutterhead and the rock support
installation area which produced spalls and slabs approximately 0.15 – 0.2 m in thickness and in
some instances when rock support could be installed prior to spalling and slabbing, fractures
would develop later. In the deepest sections of the tunnel these fractures would continue to
propagate several days to weeks behind the advancing face. This failure process is stress induced
brittle failure.

In failure class 2, the discontinuities controlled the failure process and typically these
failures occurred directly behind the cutterhead.

In failure class 3, the failure process was controlled by the interlayering of sandstone and
shale, where the sandstone was water bearing. Once exposed the shale layers would deteriorate
rapidly, failing shortly after excavation and continue to unravel if left exposed and unprotected.

A photograph of such a failure is shown in Figure 2.6.

The failures at the Navajo Tunnel No. 3 were stress induced. The horizontal nature of the sedimentary units and in some situations high water inflows contributed to the failure process at the Navajo Tunnel. These, in part, were responsible for the shape of the failed zone.

Figure 2.6: Crown and sidewall failure in the Navajo Tunnel No. 3. The tunnel is located within shale overlain by water-bearing sandstone. Picture from Sperry and Heuer (1972).
Figure 2.7: Tunnel crown failure in the Leventina-Gneiss along foliation planes with a) a general view and b) a close up of the haunch area. From Bewick and Kaiser (2009).

Figure 2.8: Horizontal to sub-horizontal fault encountered near the Bodio portal area of the Gotthard Base Tunnel (modified from Fabbri, 2005).
2.1.5.3 Leventina Gneiss along the Gotthard Base Tunnel Alignment

The Bodio Portal represents the southern terminus of the Gotthard Base tunnel and is located in the Leventina Nappe, which is composed of folded granitic gneiss (Leventina Gneiss) with a flat lying foliation in the south rotating to vertical at the Piora zone and is characterized by flat and steeply inclined ductile and brittle fault zones (Loew et al., 2000).

To start the TBM drive, a 1.2 km long tunnel had to be developed to bypass a weak rock zone in the portal area (Herrenknecht and Rehm, 2003). At the end of the bypass, a cavern was constructed to assemble the TBM. TBM tunneling through the horizontally foliated gneiss presented some crown instability which is shown in Figure 2.7.

Also during the investigation stages of the Gotthard tunnel, surface mapping results were used to project fault structures to tunnel depth. Shortly after the TBM drive was started at Bodio, an undetected horizontal fault was encountered (Pelizza and Peila, 2005). Further investigations in front of the TBM revealed an undulating sub-horizontal fault zone which the tunnel crossed several times during construction, see Figure 2.8 below. The unexpected orientation of the fault structure meant that difficult tunnelling conditions persisted longer as the tunnel followed the disturbed zone. This increased instability around the excavation and required increased rock support to advance the tunnel safely.

2.2 Review of Hydro development in the Niagara Region

The earliest record of power production from the Niagara River was in 1757, when some of the water was redirected for a sawmill (Lubar, 1989). Over the next almost 130 years, limited use of the Falls for power production was conducted, supplying local mills and industry only. Thomas Evershed proposed, in 1886, a canal and underground turbine system and thus began the search
for an economic means of transmitting the vast amount of possible power from Niagara to distant cities, such as Buffalo or even New York City.

The Westinghouse Electric and Manufacturing Company, with the help of Nikola Tesla, developed an economic transmission system (Vuckovic, 1990), which was presented at the Columbian Exposition and attracted the attention of the Cataract Construction Company (CCC) who had been searching for an economic means of transmitting power generated at Niagara Falls to Buffalo (Lubar, 1989). This transmission system is none other than alternating current, which is standard in our lives today. With an economic transmission system, the CCC began construction of a monumental power station, the Niagara Falls Power plant, which when completed in 1899 generated 50,000 hp and delivered power to the surrounding area and as far as Buffalo, NY (Brittain, 2004).

Around the same time a second power plant was built by the Schoellkof family, over top of their existing power station. The original conduits allowed for water seepage behind the new power station walls, between the rock and the power plant. Rock mass deterioration eventually caused a part of the power plant to collapse, which will be discussed later, in 1956.

Experiences from some of the earliest hydro works in the Niagara Region have played a major role in determining the geological characteristics of the area. During the construction of the first major hydro power plant in the late 1890’s, the Adams Generating Station (GS) owned by the Niagara Falls Power Company, convergence was noted in the side walls of a 6 m wide wheel pit excavated into the Lockport and Rochester Formations (Karrow & White, 2002). Damage to the tunnel lining was also found in 1908 when an inspection was conducted (Karrow & White, 2002).
In 1905, two more hydro power stations were constructed. The Canadian Niagara Power GS or Rankin GS, named after the owner of the Canadian Niagara Power Company William Birch Rankin, is located upstream of the Horseshoe Falls (Figure 1.1). The turbines are fed by the natural current, which is directed by a submerged weir. The water flows down a wheel pit 40 m below ground surface to the turbines and exits through a tunnel, 60 m long, into the Niagara River below the Horseshoe Falls. The Canadian Niagara Power GS is no longer operational. The Ontario Power GS was also constructed in 1905 by the Ontario Power Company near the base of the Horseshoe Falls. The water enters an intake tunnel at Dufferin Island (Figure 1.1), upstream of the Horseshoe Falls, and utilizes the head drop of nearly 60m to generate power before exiting directly into the Niagara River. As this station is located at the base of the Niagara River gorge, it has been the subject of gorge stability analysis over the years. This station was retired in 1999 (OPG, 2008).

In 1906 the Electrical Development Company, later the Toronto Power Company, built another power plant just downstream of Dufferin Island. The water was directed from the Niagara River by a wing dam and discharged through a tunnel which exits into the curtain of the Horseshoe Falls. It was the largest power station of its time when constructed and later it was decommissioned in 1973 (NYPA, 2008).

In 1906, Sir Adam Beck was appointed to be the first Chairman of the Hydro-Electric Power Commission of Ontario and advocated the public ownership of electrical companies in Canada (Mentzer, 2006). Sir Adam Beck’s dream of public power was realized when the first three turbines at the Queenston – Chippawa Development, now the Sir Adam Beck I (SAB I) GS, were brought on-line in 1922 (OPG, 2008). The SAB I power plant is located downstream of Niagara Falls, near the community of Queenston, so that it can utilize an additional head drop of
nearly 30m as the Niagara River flows down the Upper and Lower rapids. The SAB I GS was at design capacity by 1930, with all 10 units in operation (OPG, 2008). The power station is fed by a canal system, which takes water from the Niagara River via the Welland River and Chippawa creek, which runs through the city of Niagara Falls; see Figure 1.1. This development was the largest of its time in the world.

With increasing demand for electricity in the late 1940’s and early 1950’s, the Electrical Power Commission of Ontario undertook a massive construction project to build two diversion tunnels and the Sir Adam Beck II (SAB II) GS. The SAB II GS was brought on-line in 1954. As part of this project, a treaty between the United States and Canada was signed, limiting the amount of water that could be drawn from the Niagara River by both countries to ensure riparian rights and equal sharing of available diversion flows.

At a time when the hydroelectric power business was booming in Canada, in the early 50’s, the Americans were having difficulty getting the necessary political approval to develop more generating capacity at Niagara (Moses 1970). Five privately owned power companies were distributing power in New York State at the time and these companies had an interest in developing stations at Niagara for themselves. The power struggle was temporarily diverted in 1952 when the International Joint Commission, a Canadian – US governing body for electricity, granted approval for the development of the St. Lawrence Power project, downstream of Prescott, Ontario. When Robert Moses was appointed the chairman of the New York Power Authority (NYPA) in 1954, he headed the St. Lawrence construction project for the Americans and was also directed to develop additional power at Niagara (Moses, 1970).

In 1956, one third of the Schoellkof power plant was destroyed when the rock face behind the plant, built on the side of the Niagara gorge, collapsed due to water pressure build up
in tension cracks. Unfortunately, one worker died in the incident, but it triggered an interest in slope stability investigations in the Niagara region. The collapse represented an opportunity for Robert Moses to push through a deal for public power at Niagara (Moses, 1970). A deal was reached with the owner of the Shoellkof plant, the Mohawk Power Corporation, who had a lease for hydro production until 1971 (Moses, 1970). With the increasing demand for electricity and the shortage created by the collapse, the NYPA was in a position to develop a massive power plant which could utilize the full volume of water allowed in the treaty with Canada. With prompt funding and an experienced construction crew from the St. Lawrence project, a 3 year completion date was achievable. The debate now was over the most economic means of transporting the water from upstream of the falls to near Lewiston, New York, so as to utilize the full head drop of the upper and lower rapids, much like the SAB GS complex. The result was the construction of cut and cover conduits across the city. Other components of the power station are a pump storage reservoir covering 1900 acres, a pump storage generating facility with 12 turbines, a 2.8 million m$^3$ forbay and the power station with 13 turbines (NYPA, 2008). The facility began generating power in 1961 and at that time was the largest hydroelectric station in North America (NYPA, 2008).

Canal construction in Ontario has been an important factor in the economic prosperity of the Niagara area and the interior United States (Jackson, 1990). The first canal systems were constructed for shipping along the Great Lakes corridor, and the Niagara Escarpment presented a challenge for the Lake Erie to Lake Ontario connection. The Welland Canal first began operation in 1829 and provided this connection. The canal has since been expanded four more times, in 1833, 1845, 1887 and 1959 to adjust the route and accommodate increasing vessel sizes (Jackson, 1990). The Wellend canal runs from St. Catharines Ontario, on the shore of Lake Ontario, to
Port Colbourne, on the shore of Lake Erie. In the early years, competition with ports at Boston and New York limited the Welland canal usage, as ships could pass through to Chicago via the Erie Canal, which runs from Buffalo to Albany, or the Oswego canal, which runs from eastern Lake Ontario into the Erie Canal. It was not until the opening of the St. Lawrence Seaway in 1959 that the Welland Canal became the primary canal for shipping between Lake Erie and Lake Ontario.

The Chippawa power canal diverts a portion of the water from the Niagara River upstream of the falls as well as the Wellend river and was constructed in the 1900’s as part of the SAB I power station construction (OPG, 2008). The power canal runs from the lower Wellend river, which was reversed, and through the city of Niagara Falls and over to Queenston where it feeds into the SAB GS, see Figure 1.1.

These canals were constructed primarily in the limestone units and high horizontal stresses caused pop ups to occur during construction and operation (Karrow & White, 2002). These pop ups also occur in many quarry floors around the Niagara Region as a result of stress relief (Karrow & White, 2002).

2.2.1 Tunneling in Niagara

The capacity of the Queenston – Chippawa canal was not able to supply enough water to the planned SAB II GS of the early 1950’s. In order to feed the much needed power station, two diversion tunnels were constructed and run from Chippawa to the western edge of the St. Davids Buried Gorge near the Whirlpool area. The tunnels surface there and run the remainder of the distance to the SAB GS in an open cut canal (Figure 1.1). The tunnels utilize an effective head of
90 m (OHPC, 1953), with a total of 96 m available, due to the rapids upstream and downstream of the falls. The earlier station around the falls themselves only utilized 60 meters of head drop.

The twin, 15.5 m diameter, tunnels were constructed by drill and blast techniques from separate headings at five shafts across the city. A top heading and bench were utilized for construction purposes as shown in Figure 2.9. The primary support used was 200 mm flanged, half circular I beams with channel lagging in between (OHPC, 1953). The tunnel follows the Irondequoit Formation, a dense, massive limestone, which dips 6 m/km in a southerly direction (Rigbey & Hughes, 2007), with inclined tunnel sections of 30° at the intake and outlets (OHPC, 1953). The Lockport Formation, a strong and competent limestone could have been used for the crown of the tunnel excavation, but the higher permeability and the presence of hydrogen sulphide gas made the Irondequoit limestone a better choice, even though it is thinner. The underlying Rochester Formation is an aquitard, minimizing the inflow from the Lockport into the deeper formations.

The placement of the tunnel at this stratigraphic location did not come without some stability difficulties. The rock formations are generally constant thicknesses across the Niagara Region, with the exception of the Gasport member of the Lockport Formation which thins and thickens down dip (OHPC, 1953). Local up warping in the Irondequoit Limestone was found to exist when driving the two tunnels in the early 1950’s. The up warping was unpredictable and placed the Reynales and Neagha formations higher in the tunnel section, which caused overbreak (Figure 2.10) to occur due to two continuous shale layers in the upper 0.6 meters of the Reynales Formation (OHPC, 1953).
Figure 2.9: Top heading and bench for the water diversion tunnels of the 1950’s in Niagara Falls, Ontario (photos courtesy of Ontario Power Generation).
With accurate drilling and controlled blasting along these shale layers a neat roof could be
achieved (OHPC, 1953). Difficulty also occurred when the Neagha shale undermined the
Reynales Formation, causing overbreak in the sidewalls and haunch areas (OHPC, 1953). Rock
movements had a pronounced effect on the interbedded sandstones and shale of the Grimsby
Formation, which has thick sandstone layer and thin shale layers, caused heaving and crushing of
the rock down to the Power Glen Formation and these zones required grouting to consolidate the
rock mass – concrete liner contact (OHPC, 1953). These deformations may be better controlled
with TBM excavation; however at that time the technology was unavailable and would not have been practical given the short construction schedule which was required.

Ontario Hydro decided in the mid 1980’s to investigate the viability of constructing additional diversion and generation capacity at the SAB GS complex. Investigations and feasibility studies were performed in the late 1980’s and early 1990’s, resulting in a concept of twin 12.35 m tunnels and an underground powerhouse near the existing SAB I plant. Following these studies it was decided that a test adit would be necessary to determine if the proposed Niagara Diversion Tunnels, passing under St. Davids Buried Gorge, could be excavated by TBM methods and to further investigate the geological conditions for construction of the underground powerhouse. The main adit was approximately 570 m long, with additional adits in the proposed power house area and a trial enlargement at the end section, comparable to the proposed tunnel diameter (Huang, 1995). The adit was constructed in the Queenston Formation, starting at the base of the Niagara Gorge near the SAB GS complex and reached depths of 150 meters below ground surface.

The test adit was excavated with a road header and the trial enlargement was excavated in a series of benches and side headings (Huang 1995). Numerous sheared bedding planes were encountered in the excavations and these planes of weakness degraded the rock mass locally (Huang, 1995). It was also near these locations where spalling in the side walls and slabbing in the roof occurred, due to the high horizontal stresses with a Ko of 2.6 measured, (Huang, 1995), and bedding plane slip near the excavation, see Figure 2.11. The spalling and slabbing was in the order of 0.1 to 0.5 m deep and within the trial enlargement these deformations occurred soon after excavation and continued after rock bolts and mesh were installed, up to three months after excavation had ceased (Huang, 1995).
The trial enlargement showed that TBM excavation was possible for the Niagara Tunnel Project, but that rock support would have to be installed immediately behind the cutterhead to prevent large overbreak from occurring and damaging the machine (Huang, 1995).

Figure 2.11: Photo of the trial enlargement in the test adit for the Niagara Tunnel Project. Note the sheared bedding plane half way up the face of the excavation and the associated sidewall slabbing (photo courtesy of Ontario Power Generation).
2.3  Geological Evolution of the Niagara Region

To fully understand the geological evolution of the Niagara Region it is worthy to look back at the historical accounts and hypothesis of the early explorers in the area. These explorers observed nature as it stood before them and through their early observations and hypothesis created the foundation of our modern thinking. It was the Geological Survey of Canada, a government branch of the Province of Canada directed by Sir William Edmond Logan and formed in 1842, which pioneered and bolstered our understanding of geological processes (Zaslow, 1975). As Sir Charles Lyell once wrote in his *Principles of Geology*, published in 1833 (Zaslow, 1975);

“We shall adopt a different course, restricting ourselves to the known or possible operations of existing causes; feeling assured that we have not yet exhausted the resources which the study of the present course of nature may provide, and therefore that we are not authorized, in the infancy of our science, to recur to extraordinary agents. We shall adhere to this plan … because history informs us that this method has always put geologists on the road that leads to truth, - suggesting views which, although imperfect at first, have been found capable of improvement, until at last adopted by universal consent. On the other hand, the opposite method, that of speculating on a former distinct state of things, has led invariably to a multitude of contradictory systems, which have been overthrown one after the other, - which have been found incapable of modification, - and which are often required to be precisely reversed.”

Based on Charles Lyell’s advice, the review of the current literature shall be used as a foundation for which new ideas for the glacial evolution of the Niagara Region will be presented

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2 The geological evolution of Niagara, particularly the stress regime, is presented in Perras & Diederichs (2007) which is included in Appendix B.
later in this chapter. These “improvements” will be “imperfect”, but it is hoped that by combining an understanding of glacial processes and rock mechanics, a better understanding of the evolution of the Niagara Region will be presented and promote future study.

The focus of the geology section is on the near surface strata, which has been observed first hand, in an un-weathered state, via the Niagara Tunnel Project. This includes the Queenston Formation of the Upper Ordovician and the Silurian sequence up to the Lockport Formation, as shown in the stratigraphic section in Figure 2.12. For completeness, the strata from the Precambrian to the Queenston and those Formations above the Lockport, namely the Salina, are described briefly only.

The impact of ice sheet movement would be greatest on these near surface formations, at <200 m in depth according to Mazurek (2004), although according to Boulton and Caban (1995b) hydrofracturing could be theoretically possible to depths reaching 250 - 400 m. The Niagara Escarpment and the various gorges which cut down through the escarpment provide localized areas where greater depth of ice sheet influence on the bedrock is possible, as will be shown in the later sections of this thesis.

### 2.3.1 The Rock Record

The Taconic Mountain range, later forming the Appalachians, provided the eastern and southern most limits of the Appalachian basin or Allegheny trough, a foreland basin, as seen in Figure 1.6. As this mountain chain began to form in middle Ordovician time, the shallow waters of the basin allowed for abundant life to create limestone units of the Black River and Trenton groups, in a shallow shelf environment.
<table>
<thead>
<tr>
<th>Age Group</th>
<th>Formation</th>
<th>Description</th>
<th>Avg. Thickness (m)</th>
<th>Symbol</th>
</tr>
</thead>
<tbody>
<tr>
<td>Middle Silurian</td>
<td>Lockport</td>
<td>Grey crystalline dolomitic limestone</td>
<td>16.8-20.3*</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Decew</td>
<td>Crystalline dolomite and grey mudstone</td>
<td>2.1-4.0*</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Rochester</td>
<td>Dark grey calcareous shale dolomite interbedded</td>
<td>18.7</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Irondequoit</td>
<td>Grey to reddish dolomitic limestone</td>
<td>3.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Reynales</td>
<td>Light grey crystalline dolomite</td>
<td>3.3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Neahga</td>
<td>Green shale</td>
<td>2.1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Thorold</td>
<td>White sandstone with shale interbd.</td>
<td>2.7</td>
<td></td>
</tr>
<tr>
<td>Lower Silurian</td>
<td>Grimsby</td>
<td>Green, irregularly bedded sandstone with red shale interbeds</td>
<td>13.3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Power Glen</td>
<td>Grey shale to white calcareous sandstone</td>
<td>9.7</td>
<td></td>
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<tr>
<td></td>
<td>Whirlpool</td>
<td>Light grey cross bedded sandstone</td>
<td>6.0</td>
<td></td>
</tr>
<tr>
<td>Ordovician</td>
<td>Queenston</td>
<td>Red siltstone and argillaceous limestone</td>
<td>335*</td>
<td></td>
</tr>
</tbody>
</table>

Figure 2.12. Stratigraphic section (Perras and Diederichs, 2007) of the formation which outcrops along the Niagara Escarpment near Niagara Falls, Ontario, Canada. Thicknesses are those observed in the Niagara Tunnel Project or as indicated (*) by Haimson (1983).

Due to the shallow waters the environment was constantly being agitated by wave and current motion, which disturbed and broke up much of the shell fragments and corals forming lime mud (Stearn et al., 1979).

During Ordovician time, the east coast of North America was a much different place then it is today. Volcanic rocks in the Northern part of the Appalachians indicate that subduction was
occurring and island arcs were being formed. On the coastal side of the Appalachians the sediment supply, from the mountain building, increased and became more argillaceous westward and displaced the limestone of the shallow shelf. The sediment volume increase created sandstone units close to the edge of the mountains and formed shale units further inland. These are predominately the Blue Mountain, Georgian Bay and Queenston Formations (Mazurek, 2004). Stearn et al. (1979) called these formations, plus those of Silurian clastic sediments presumably up to the Grimsby and Thorold Formations, the Queenston clastic wedge. It is here that we start a more detailed description of the deposition and diagenesis of the remaining strata of the Niagara Region.

The Queenston Formation, which gradationally overlies the Georgian Bay Formation, is Upper Ordovician in age and is part of a compound deltaic, shallowing upward sequence (Stearn et al., 1979) of a semiarid region (Brogly et al., 1998). The Formation is a red and grey argillaceous mudstone with occasional siltstone and sandstone interbeds and fossiliferous siltstone to limestone beds near the base (Rigbey et al., 1992 and Brogly et al., 1998) and is greater than 300 m thick in the project area. The sediment was provided by rivers flowing down from the Taconic highlands and with the lack of plant life the landscape was a red, barren alluvial plain (Stearn et al., 1979). Moving away from the highlands toward the Niagara Region during the close of the Ordovician, the formation is mostly composed of red mudstone with green-grey shale which is petrographically and structurally similar to the red mudstone (Brogly et al., 1998). The mudstone is a lagoonal deposit, which was periodically inundated by sea water (Stearn et al., 1979). This inundation into areas of restricted flow allowed for the concentration of evaporite minerals in the shallow waters and as these areas began to dry out, salts and other evaporite minerals began to form. It is in this manner that the Queenston Formation became a salt rich
sedimentary unit, which exhibits highly connate ground water today (Rigbey & Hughes, 2007). Gypsum is present throughout the formation and Brogley et al. (1998) reports that it occurs as nodules or thin laminae which are both observable in the Niagara Tunnel, where a sharp increase in nodules, gypsum/calcite filled burrows and laminae exist below elevation 55 m.a.s.l.

The greenish – grey banding in the Queenston Formation is formed as a secondary process called reduction. Percolating ground water may be the cause of the reduction of the iron molecules, which makes the Queenston a characteristic red colour, to the greenish – grey colour seen today in horizontal beds (Telford, 1975) and vertical joints (Figure 2.13). The process is simply a chemical reaction, whereby the Fe(III), or ferric iron, of the red Queenston is reduced to Fe(II), or ferrous iron, of the greenish – grey bands, by the addition of water and the reaction is principally controlled by the pH and Eh of the environment.

Figure 2.13. Horizontal reduction banding within the Queenston Formation (photo courtesy of Ontario Power Generation).
At the close of the Ordovician and prior to the start of the Silurian, an erosional period occurred where the upper-most part of the Queenston Formation was removed (Mazurek, 2004) prior to the deposition of the Whirlpool Formation. This represents a significant disconformity between the Queenston and Whirlpool formations. Three Groups, the Cataract, the Clinton and the Albemarle, overlay the Queenston Formation.

The Cataract Group is Lower Silurian in age and was deposited in deltaic and shallow marine environment (Currice & Mackasey, 1978). It includes the Whirlpool, Power Glen and Grimsby Formations. The Whirlpool sandstone, which disconformably overlies the Queenston (Mazurek 2004), represents a transgressional state of a sea that washed and rounded quartz sand grains (Stearn et al., 1979). Tesmer (1981) also suggests that portions of the Whirlpool were deposited sub-aerially based on frost round quartz grains, which could be windblown beach sands. The Whirlpool Formation has been observed to be on average 6 m thick, at the site of the Niagara Tunnel Project.

The Power Glen Formation is a gray shale with white calcareous sandstone interbeds. It can be broken down into two units; the upper unit being a light grey sandstone with grey interbeds of shale, and the lower unit is predominately grey shale with interbeds of light grey sandstone. The formation was observed to be roughly 10 m thick at the tunnel, with sub-units of equal thickness, generally speaking. The deposition of this shale rich formation marks an increase in sea levels. With minor fluctuations in the sea level the depositional material alternated between sand, clay and calcareous shell fragments, forming a shale sequence with interbedding of sandstone and limestone (Winder & Sanford, 1972).
The Grimsby Formation consists of thin beds of red shale, 25 to 200 mm thick, interbedded with red fine grained sandstone. Ripple marks were observed in hand samples collected at the Niagara Tunnel Project, suggesting deposition in a sub-aqueous environment. On the other hand, mud cracking has also been observed, suggesting alternating flooding and exposure. This is consistent with Winder and Sandford (1972) who state that minor fluctuations in the sea level provided alternating depositional material, from sand, clay and calcareous shell fragments to form the Power Glen and Grimsby Formations. The Grimsby Formation was observed to be on average 13.3 m thick at the Niagara Tunnel Project.
The stratigraphic interval between the Whirlpool and the Reynales, shows little break in the sedimentation, however, lateral gradational changes can be observed traveling north to south (Stearn et al., 1979). A stratigraphic section (Figure 2.14) from Stearn et al. (1979) shows a correlation between the formations and indicates name changes where applicable. Note that the Cabot Head Shale is now known as the Power Glen Formation.

The Clinton Group is Middle Silurian in age and comprised of the Thorold, Neahga, Reynales, Irondequoit and Rochester Formations, which were deposited in a shelf edge environment (Winder & Sanford, 1972). Reworked Grimsby detritus formed the Thorold Sandstone as sea levels increased. Tesmer (1981) also suggests that portions of the Thorold are windblown beach sands because of frost round quartz grains. Complex interbedded shale bands can be observed in the tunnel walls of the Niagara Tunnel Project, as shown in Figure 2.15, suggesting frequent flooding. It is a relatively thin unit, on average 2.7m thick, as observed in the tunnel.

Figure 2.15. Complex interbeds of shale within the Thorold sandstone (photo courtesy of Ontario Power Generation).
With continuing sea level changes in an intertidal and lagoonal environment, the Neahga and Reynales Formations were deposited (Winder & Sanford, 1972) as the sea levels decreased. The Neahga Formation is a thin, 2.1 m thick, fissile green shale layer which marks the transition from siliclastic to the calcareous rich Reynales Formation. The Reynales Formation is a light grey crystalline dolomite which was on average 3.3 m thick at the Niagara Tunnel Project.

The Reynales contains two continuous shale layers in the upper 0.6 m of the formation (Figure 2.16), which were both observed in the Niagara Tunnel Project, as shown in the previous hydro tunnels from the 1950’s (OHPC, 1953).

The Irondequoit Formation is a crinoidal dolostone, with occasional bioherms, which can be seen (Figure 2.17) to be draped by the dolomitic muds forming the Rochester Formation. The Irondequoit is a grey to reddish dolomitic limestone which was observed to have an average thickness of 3.0 m at the Niagara Tunnel Project.

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

Patch reefs, forming the lower Lockport Formation and regional reefs, forming the upper Lockport and Guelph Formations (Tesmer, 1981), completely surrounded the Michigan Basin and allowed for the precipitation of evaporates of the Salina Formation (Sanford, 1968), which overlies the Lockport.

This is the stratigraphy of the Niagara Region, which is exposed along the Niagara Escarpment. The names of the formations change from location to location, especially those in the United States, due to gradational changes in the units, pinching out, and different type localities for the same unit.
Figure 2.16: Two continuous thin shale layers, less than 20 mm thick, in the upper Reynales Formation as observed in the Niagara Tunnel Project (photo courtesy of Ontario Power Generation).
Figure 2.17. Picture from the Niagara Tunnel Project showing a patch reef protruding up into the Rochester Formation (photo courtesy of Ontario Power Generation).

2.3.2 Review of Glacial History

There are over 17 major continental glaciations recorded in the sedimentary record of the Pleistocene. The earliest recognized stages of the Laurentide are the Illinoian and the Wisconsin, oldest and youngest, respectively (Menzies, 2002). The Wisconsin is the most studied and recognized, as tills from this stage are relatively un-weathered and almost un-modified (Hough, 1958). The pre-Wisconsin ice of the Laurentide ice sheet is estimated to range between 2000 – 3000 m thickness near Hudson Bay, and during the Wisconsin reduced thicknesses of 500 – 1300 m have been reported by Menzies (2001), in the Niagara Region.
The interglacial period between Illinoian and Wisconsin time is called the Sangamon and is marked by paleo-soils between till deposits. The Laurentide ice sheet is estimated to have been 35% of the world ice volume at the time and it covered much of Canada and down into the mid west of the United States at its maximum as shown in Figure 2.18. The Cordilleran ice sheet covered British Columbia, the Yukon and most of Alaska during the Wisconsin age. There are five major glacial stages during Wisconsin time, marked by Marine oxygen isotopes, where the Laurentide ice sheet advanced and retreated according to cooling and warming cycles (Menzies,
These advances and retreats would have had a significant impact on the topography of the Niagara Region.

The main topographic features of the Niagara Region, namely St. Davids Buried Gorge, were in existence prior to mid-Wisconsin time (Hobson & Terasmae, 1969) and the Niagara Escarpment shows evidence of existence prior to the last ice advance in Wisconsin time (Hughes, 1958). According to Pengelly’s 1996 paper, the Niagara River gorge is post-Wisconsin. These features hold keys to the glacial history of the Niagara Region, and as I will explore later, these features may have had major influences on the bedrock structure which has developed.

At the southern limit of the Laurentide ice sheet many large valleys have been recognized (Hooke & Jennings, 2006), and in the Great Lakes area, many filled valleys have been identified and reported as early as 1890 by Spencer. Spencer (1890) described an Ancient Laurentian Valley, were the Great Lake basins were eroded by a river, he named Laurentian, prior to the Pleistocene period. If this is the case, then the Great Lake basins existed prior to Pleistocene ice sheet advance over southern Ontario, and Spencer (1890) also indicates, in his map of the drainage pattern (Figure 2.19) that the Niagara Escarpment also existed at this time. These features present interesting possibilities for tunnel valley formation, a valley carved out by high pressure water underneath a glacier, in the Niagara Region. Whether or not the valleys of the Niagara Region are tunnel valleys, the glacial impact on the drainage pattern is significant.
2.3.2.1 Bedrock Erosion and Deformation by Glacial Activity

Bedrock erosion due to glacial movement is a slow and gradual process. Erosional mechanisms fall into two categories, surface and sub-surface. Both processes occur in the same local subglacial regimes and although they are not dependent on each other, their combined efforts enhance the erosion process. Melt water plays a key role in helping erosion and specifically fluctuations in temperature are critical in the quarrying process. A byproduct of the ice pressure is glacial melt water which flows along the ice – bedrock, ice – sediment contact or within the sediment and or bedrock. This melt water actively erodes bedrock along fracture planes and surfaces and helps to round rough edges left from quarrying, joints, or channels.
2.3.2.1.1 Glacial Erosion of Near Surface Bedrock

The act of abrasion is a mechanical wearing or grinding away of a rock surfaces by friction and/or debris impact (Bates & Jackson, 1983). In glacial ice sheets, debris entrained in the ice abrades the surface of the bedrock as the ice moves over the surface. This action creates many features which are recognizable on exposed bedrock surfaces today.

Grooves and striations are common linear features on bedrock surfaces and are formed when hard entrained debris is dragged across the bedrock. The grooves and striations can be any size, formed from coarse sand or large boulders, producing similar sized features respectively. These features run parallel to ice movement.

Chatter marks are formed by impact of debris onto the bedrock surface and result in brittle fracturing of the bedrock due to the point load. It is thought that the piece of debris rotates within the ice during the impact, such that it does not drag down ice and create a groove. The chatter marks occur congruent with other abrasion marks. As with grooves and striations, chatter mark size is related to the size of the impacting debris.

Abrasion in itself tends to lead to shallow erosion, but none the less plays an important role in smoothing and shaping the bedrock surface. Abrasion often leaves a polished look to the bedrock surface and this is further enhanced by melt water polishing. Abrasion and debris impact on the upland area of the escarpment could be important influences in starting the erosion process by developing an uneven bedrock surface, which is later utilized for melt water erosion.

The process of quarrying involves the removal of larger rock fragments and blocks and often leads to an area of heavily fractured rock. It is necessary to have an undulating bedrock surface or other uneven surface for the process to begin. As the ice sheet passes over a bump in the bedrock surface, the ice becomes detached from the bedrock. The gap creates a void for
subglacial melt water to accumulate. As the water flows into the void, the pressure drops and the water re-freezes. Then the ice advances again, the void reopens and the process is repeated. This process removes blocks and creates fracture networks, which leads to further quarrying. In Niagara the escarpment edge would provide a location for quarrying and plucking to occur.

Mobile sediment beds are a layer of till between the bedrock and ice base. The mobile sediments can move either as one mass or deform under the ice load. In the previous case, it is believed that the sediment moves as a slurry and becomes highly mobile under ice and internal pore water pressures. The pore water pressure plays an important role in maintaining movement of the sediment base. If the pore pressure drops, then movement will decrease due to increased frictional resistance at the bedrock contact.

Mobile soft sediments are a key component to erosion in the Niagara Region. As continental ice sheets, with thicknesses less than 1000 m, advanced over the region, the presence of a mobile soft sediment layer would both increase erosion and the mobility of the ice sheet and locally generate ice streams. This would increase the ability of rock erosion, especially where reduced ice thicknesses have been reported by Menzies (2002).

Glacial melt water plays an important role in all erosion aspects and on its own can carve out and widen bedrock fractures and smooth rock surfaces. The glacial melt water is under high pressure at the base of the ice sheet and as it passes over the bedrock surface, cavitation, fluid-stressing and corrosion processes act to wear and smooth the surface. Different erosional marks can occur as the water passes around obstacles (transverse forms), or use existing channels, fractures and topography (longitudinal forms) or of random occurrence (non-directional forms) as shown in Figure 2.20. These landforms develop as a complex interaction between topography, bedrock structures and geometry, subglacial stresses and hydraulics and evacuation routes.
Figure 2.20: Subglacial melt water erosional landforms (from Kor et al., 1991).

These landforms occur mostly at the bedrock surface, with the exception of cavetto, furrows and potholes which can widen existing structures and down cut into the bedrock surface to variable depths. Potholes in particular appear to occur at major breaks in slope (Menzies, 2002) and may be found in the Niagara Region near the escarpment and valleys.

A detailed exploration of bedrock surface erosion is outside the scope of this research and the reader is referred to the text by Menzies (2002) for a thorough description of glacial processes, which includes many references to leading researchers in this field. As this research is
focused on underground excavation stability, processes which affect joint development and other yielding process which can occur due to glacial ice sheet activity will be explored in more detail.

2.3.2.1.2 Subsurface Glacial Bedrock Damage

Recently more work has been done to understand how glacial ice sheets can create structure at depth in bedrock. Several processes are plausible, namely shear over stressing, hydraulic fracture initiation and tensile over stressing and have been described by Boulton and Caban (1995b). Intuitively the impact of glacial ice sheets decreases with depth from the bedrock surface and Boulton and Caban (1995b) propose that depths of 250 - 400 meters may be influenced. These processes could be responsible for changing hydrological parameters during ice sheet loading (Boulton & Caban 1995a). The damage is induced by either movement or direct loading (Boulton & Caban, 1995b) and will be further explained in the following sub-sections.

2.3.2.1.2.1 Shear Over Stressing

Shear failure can occur by both ice sheet movement and direct ice sheet loading. The ice movement is a thin skinned phenomenon (Boulton & Caban, 1995b) and direct ice sheet loading can cause shear failure near surface, as well as at greater depths. In both cases the pore water pressure, ice sheet dynamics (ice thickness, permafrost depth and drainage conductivity), bed strength properties and in-situ stress field play an important role.

Near surface, the drag created as the ice sheet moves over the land surface can create slip failure plains in the bedrock. Typically this is only a very shallow deformation process, although Boulton and Caban (1995b) suggest that considerable depths could be reached if very high pore water pressures are reached, possible due to permafrost reaching the joint fracture and karst
systems near surface thereby reducing the conductivity. For shear failure to occur, the rock mass strength must be overcome and this can be estimated to occur using Equation 2.1, when:

\[ \frac{\tau_s - C}{\sigma'_n} \geq \tan \phi \]

Where \( \tau_s \) is the shear stress on a given plane, \( C \) is the cohesive strength of the material, \( \sigma'_n \) is the normal stress on the given plane and \( \phi \) is the angle of internal friction, as given by Boulton and Caban (1995b).

The orientation of the potential shear plane is related to the stress field, which has been modified from the in-situ stress field by ice loading. As the ice sheet advances over the bed the vertical stress is increased and locally near the bed the vertical stress becomes the dominant stress \((\sigma_1)\). The depth to which the vertical stress is dominant will be controlled by the in-situ stress field and the rock mass properties.

It is difficult to distinguish today what portion of in-situ stress developed from different mechanisms (i.e. tectonic, ice loading). The zone of potential shear failure is isolated to below the ice sheet itself (Boulton & Caban, 1995b) and the slip planes will generally dip down ice since this is the direction of least resistance.

The effect of permafrost on shear failure would be through reducing the effective stresses at depth by increasing the water pressure, thereby reducing the resistance to slip. Effectively this is reducing the normal stress \((\sigma'_n)\) from equation 1, thereby increasing the left hand side value and making the rock mass more susceptible to shear failure.
2.3.2.1.2.2 Hydraulic Fracturing

Hydraulic fracturing or hydrofracturing is a failure mechanism that can be caused by ice sheet loading. In its pure sense, it is the buildup of fluid pressure, which decreases the effective stress until the tensile strength has been overcome (Davis & Reynolds, 1996). Hydrofracturing has been used by the oil and gas industry to increase permeability at depth by injecting water or sand water slurry into reservoirs to fracture the rock and create more flow paths. Hydrofracturing is also used in determining the in-situ stress field at depth through bore holes (Haimson & Cornet, 2003). The principles of hydrofracturing for in-situ stress field measurements can shed some light on how hydrofracturing can occur under ice sheets and the probable orientation of the tensile failure.

In the method of in-situ stress field measurement, a section of a drill hole is isolated with a rubberized packer, which is inflated above and below the desired depth. Once the zone is isolated, water is injected into the isolated zone and the pressure is allowed to build up. Once the pressure or induced stress reaches the drill hole rupture strength a fracture will be generated parallel to the maximum stress (Kim & Franklin, 1987). This pressure can then be used to determine the stress which was overcome, and by subsequent opening and closing, by pressure increases and decreases respectively, on the same fracture, this stress can be checked.

In a similar manner during water pressure build up under an ice sheet, hydrofracturing is possible and will generate joints and fractures parallel to the maximum stress and perpendicular to the minimum stress. These joints would probably be rough, due to the rapid opening and also could contain plumose structures from repeated opening and closing cycles. A plumose structure is a texture on the joint surface with a pattern similar to that shown in Figure 2.21.
When analyzing joint patterns and trying to determine the mechanism of origin, remember that the stress field is altered due to the direct ice sheet loading. As can be seen in Figure 2.22, the maximum stress rotates into the vertical direction under the ice sheet. If conditions prevail to hydrofracture the rock mass, the joints generated will typically be in a vertical orientation. As the ice advances and retreats in a cyclic manner, however, joint orientations could be variable, switching between vertical and up ice dipping according to Boulton and Caban (1995b) and thus a long standing position of the ice toe would more likely
generate the one particular orientation at a specific site. For the up ice dipping joint configuration to be activated near the ice sheet toe, where more drainage is possible, there must be thicker ice present to generate sufficient pressure for hydraulic fracturing to occur.

![Figure 2.22: Maximum stress rotation at the toe of an ice sheet. Initial stress conditions were set to hydrostatic and then ice load was applied using additional material. Model results from Phase2.](image)

Figure 2.23: Zones of possible tensile overstressing and hydraulic fracturing at the Niagara Escarpment. The permafrost can help extend the fracture development beyond the edge of the escarpment by increasing water pressures.
2.3.2.1.2.3 Tensile Over Stressing

In hydrofracturing, the tensile strength of the rock is overcome by excessive pore water pressure. Tensile failure can also occur from direct ice loading at topographic elevation changes and at the toe of the ice sheet due to flexural bending. At these locations, the ice load is concentrated, which can cause direct crushing, but at further distances from the edge, tensile failure will occur as the load causes deflection of the bedrock surface. The process of deflection can be thought of as a beam which is being pushed down on one end. The top of the beam experiences tensile stress and the bottom experiences compressive stress.

At the topographic extremes of the Niagara Escarpment, and the lip of St. Davids Buried Gorge, these tensile type failures should be prominent and possible locations of occurrence are shown in Figure 2.23. Tensile fractures, where the ice sheet lifts off the ground surface, would radiate normal to the curvature of the ice and create sub-vertical to vertical fractures. At the escarpment face, tensile fractures could develop parallel to bedding as the hydraulic pressure jacks open fractures along the bedding planes. Along the Niagara Escarpment such features may be exposed by recent escarpment erosion processes which are described by Barlow (2002).

2.3.2.1.3 Tunnel Valley Processes

Tunnel valleys or tunnel channels are created underneath the margin of ice sheets by the catastrophic release of melt water. These topographic features are characteristically elongate and irregular deep depressions cut into the bedrock (Jorgensen & Sandersen, 2005).

The authors of tunnel valley and tunnel channel research use the terms interchangeably, although Fisher et al. (2004) does make a distinction, as shown in Figure 2.24.
The term tunnel valley refers to a valley created by lateral and vertical movements of subglacial rivers. On the other hand a tunnel channel, according to Clayton et al. (1999), is a valley created by bank full subglacial rivers. The basic principles of formation and the characteristic features of both tunnel valleys and tunnel channels are the same, with the exception of the volume of water flowing through the system at a given time. It is conceivable that such systems would be re-used time and again as the tunnel represents the least resistant flow path. Therefore after the first formation of the tunnel under the ice sheet, the flow would probably not be bank full. Based on this idea the term tunnel valleys will be used for the remainder of this discussion.
It was Spencer’s work from 1890 which led to the discovery of multiple channels under the glacial sediments of the Great Lake Basins in Ontario. Spencer describes these river channels as the ancestral Laurentide Valley drainage system, which he believed carved out the Great Lake Basins. Specifically, he describes drainage out of the Erie basin, via the present day Grand River and other buried tributaries, which have valley bottoms of much greater depth than the Erie Basin, as much as 30 meters. He also describes other buried valleys which were later found to exhibit tunnel valley characteristics.

The characteristics of tunnel valleys are undulating bottom topography, an upward-adverse to river flow general slope trend, or often convex up longitudinal profiles (Hooke & Jennings, 2006). This irregular profile is most likely due to the surging nature of the flow through the valley and the repeated use of the valley for rapid drainage. Other characteristics are abrupt termination, sometimes at the apex of an outwash fan, with rectilinear patterns (Jorgensen & Sandersen, 2005).

In the recent past, tunnel valleys have been recognized in south-central Michigan and these valleys once thought to be river channels, now play an important role in understanding the evolution of subglacial and ice-marginal landscapes (Fisher et al., 2004). These tunnel valleys range in width from 180 m to 1000 m, and are 10 m to 70 m deep, as reported by Fisher et al. (2004), after Wright (1973) and Clayton et al. (1999). Tunnel valleys commonly occur as networks or at regular distances apart in the same region, and Hooke and Jennings (2006) suggest this is because a certain minimum drainage area is required. Menzies (2002) states that linear erosion features, such as troughs, fjords and tunnel valleys are probably formed under such linear features as ice streams. This could be another reason for the regular spacing of tunnel valleys suggested by Hooke and Jennings (2006).
There is evidence that suggests the Laurentide Ice Sheet drained by catastrophic events at its ice margins. Examples of this are tunnel valleys in the Oak Ridges Moraine area of southern Ontario (Russel et al., 2002) and tunnel valleys in south-central Michigan (Fisher et al., 2004). It is not inconceivable to think that if conditions existed elsewhere along the Laurentide Ice Sheet margin; they could also have formed in the Niagara Region.

The processes of tunnel valley formation and other sub-surface deformation processes will be explored in greater detail in the following section, where the focus will be on the Niagara Region specifically and numerical modeling will be used to test these theories.

2.3.3 Review of Niagara River Erosion

The erosion of the Niagara River gorge was initiated at the end of the last glacial period as part of a multi-outlet glacial lake system (Calkin & Brett, 1978). Two main glacial lakes, Lake Tonawanda to the east of the existing river, and Lake Wainfleet to the west, are associated with glacial Lake Algonquin flooding and the Nipissing Rise (Pengelly et al., 1997). Lake Tonawanda had multiple outlets, at Holley, Medina, Gasport, Lockport and Lewiston, flowing over the existing Niagara Escarpment (Figure 2.25), and as lake levels fluctuated, the flow was focused to the Lewiston area, where the Niagara River Gorge began forming. Lake Wainfleet was correlated to Lake Tonawanda by Tinkler et al. (1994), which also had multiple drainage points during exceptional highstands (Figure 2.25), but primarily drained east to the Niagara River (Pengelly et al., 1997).
Figure 2.25: Extent of glacial lakes Wainfleet and Tonawanda with important glacial history sites indicated by numbers. From Pengelly et al. (1997).

The Lake Erie water levels at the end of the last glacial period would have controlled the erosion of the Niagara Gorge. The discharge of the Niagara River into glacial Lake Irondequoit was controlled by two sills, ridges of rock which the river must flow over. The location of these sills, the Fort Erie/Buffalo sill (12) and the Lyell/Johnson sill (18) are shown in Figure 2.25. The Fort Erie sill, located at Buffalo USA/Fort Erie Canada, controlled the Niagara River flow during most of the erosional period. However, due to isostatic rebound, the Lyell/Johnson sill, located 2km downstream of the present Niagara Falls, become higher than the Fort Erie sill for a short period until it was eroded. The discharge from Lake Erie controlled the erosional rate of the Niagara River and this must have been fairly consistent for the first ~4km of erosion, because the gorge shape is roughly the same (Pengelly et al., 1997). The first major change in the cross section of the gorge occurs at the Niagara Glen, where the present river flows through a narrow cross section and higher terraces occur within the overall gorge (Tinkler et al., 1994).
terrace, at higher elevations, mark a higher volume of water flow, which subsequently decreased. The timing could be coincident with the North Bay outlet, of the upper Great Lakes, which would decrease the outflow from Lake Erie, and as the glacial melt waters decreased in volume, a smaller cross sectional area of gorge would be eroded. An interesting feature of the Niagara gorge, as well as the St. Davids Buried Gorge, is the fact that the upper limestone cap rock has been rounded, possibly by glacial activity (Spencer, 1907). Spencer suggests that the position of the gorge is related to a shallow glacial valley, which was subsequently deepened by river erosion following melting of the ice sheet. The erosion of the Niagara River gorge to its present location at Niagara Falls is reported, by Pengelly (1997), to have taken 10 000 to 12 000 years. The total length of the gorge is 11 km and the present erosional rate at the Horseshoe Falls is roughly 0.3 m per year. This erosional rate is much reduced as compared to the past due to water diversion for hydropower.

2.3.4 The Development of the Niagara Escarpment

The Niagara Escarpment is a cuesta-type landform, running roughly north-south which extends from Northern New York State, along the western shore of Lake Ontario and up to the tip of the Bruce Peninsula, where it is submerged under Lake Huron. The escarpment is capped with the resistant limestone of the Lockport-Guelph Formations in the Niagara Region or the Amabel Formation further north. The current erosional process of the escarpment is due to creep of the underlying shale formations (Barlow, 2002). Evidence supporting this assessment was published by Barlow (2002) based on a conference paper by Hewitt (1997) as;
1) cambering of the cap rock towards the scarp face
2) progressive dilation of joints
3) ‘jostling’ of detached blocks
4) crevice cave formation via backward rotation
5) un-weathered blocks and talus below the cliffs
6) absence of fluvial action parallel to scarp face.

The present day deformation process of the Niagara Escarpment may be different than the historical due to glacial drift build up at the base of the escarpment disrupting the pre-glacial drainage patterns (Barlow, 2002). The origins of the Niagara Escarpment are still not clearly defined in the literature. It may be due to river erosion parallel to the face and this drainage pattern was disturbed by glacial activity or it may have formed as a wave cut cliff in an early glacial lake (Barlow, 1995). In either case, the rock layers have played an important part in preserving this topographic feature.

2.3.5 Review of St Davids Buried Gorge Erosion

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

- Non-continuously falling thalweg
- Undulating longitudinal profile
- Rectilinear to slightly sinuous surface outline
- Abruptly terminating
- Horizontal and sub-horizontal shear surfaces with less than 60 cm movement at depth
- Steep valley side according to Spencer (1907)
- Down cuts into the Queenston Formation, except at saddle point

A number of theories regarding the location of St. Davids Buried Gorge have been postulated, although few have been published. Spencer (1907) indicates that the limestone crest of the gorge in the Whirlpool area has been rounded and suggests that a pre-existing valley was scoured by glacial activity and subsequently eroded by melt water. Others have suggested that it may be a fault; however this was disproved by the Niagara Tunnel Project, which only encountered horizontal and sub-horizontal sheared surfaces with less than 60 cm of movement, as shown in Figure 2.26. These sheared surfaces are a result of the erosion of the gorge in a high
horizontal stress environment as reported by Perras and Diederichs (2007). A macroscopic illustration of the interaction between horizontal and sub-vertical shears is given in Figure 2.27.

Figure 2.26: Sheared zone underneath St. Davids Buried Gorge as observed in the Niagara Tunnel. A green reduction band and a vertical joint are shown to be offset. The sheared zones have localized pockets of enlarged shearing as shown in photo and follow thinner sheared surfaces of longer extent (photo courtesy of Ontario Power Generation).
2.3.6 Structural Features of the Niagara Region

In the Niagara megablock, as defined by Sanford et al. (1985), there are three major physiographic features, which control the structural style within their immediate vicinity, namely the Niagara Escarpment, the Niagara River Gorge and St. Davids Buried Gorge. These three features intersect to form a large triangular block of rock, of which the Queenston Formation forms the base.

There are considerable stress relief fractures along the gorges and the escarpment, due to the tensile zone created by the adjustment in the horizontal stresses around these topographic features.

A weathered fracture network near the bedrock surface is pervasive through the Niagara Region (Novakowski and Lapcevic, 1988). Bedding plane fractures are predominant in the Niagara Region, with vertical jointing being widely spaced (Novakowski and Lapcevic, 1988).
Nearing the physiographic features mentioned above, vertical and horizontal jointing increases due to a zone of tensile stress relief (Novakowski and Lapcevic, 1988).

In the Queenston Formation slickensided shears are ubiquitous features (Russell and Harman, 1985) that have been observed in both weathered and core samples. In addition to these ubiquitous shears, more widely spaced horizontal shears have been observed in the upper elevations of the Queenston Formation in a test adit driven for the Niagara Tunnel Project. These sheared bedding planes have a spacing of 8 – 10 m and are sometimes filled with silt (Rigbey et al., 1992). The silt filled discontinuities are close to the Whirlpool contact and decrease with depth (Rigbey et al., 1992). They appear to be relatively unaffected by the distance from the gorges (Rigbey et al., 1992), suggesting that they are not related to gorge erosion.

Other structures in the Niagara Region do exist, such as inclined shear planes in the Rochester and Neagha formations, but only several have been reported (Mazurek 2004) and only one was observed in the Neagha formation via the Niagara Tunnel Project. These features may be formed due to direct loading – unloading by an ice sheet (Karrow & White, 2002) or drag forces, since there is little evidence of tectonic influences after Middle Silurian time in the Niagara Region (Stearn, 1979). A report by Novakowski and Lapcevic (1988) suggests another lineament, at the base of the Rochester Formation, as determined by isopac mapping. Sanford et al. (1985) reports a fault network. These are the only other lineaments reported in the literature.

For the Niagara Tunnel Project four sub-vertical to vertical joint sets and one horizontal set, parallel to bedding, were identified within the Niagara Region. The strikes of the sub-vertical sets are 005°, 045°, 085° and 135° from north. The strikes are roughly parallel to the major topographic features of the region, namely the Niagara Escarpment, St. Davids Buried
Gorge and the Niagara River Gorge. The joints are typically short, planar, slightly rough, and can contain gypsum / calcite infilling.

A joint set was observed near the upper elevation of the Queenston Formation, which is laterally continuous over more than 20 meters, as it intersected the tunnel wall to wall at an angle of 45°, with an orientation of ~117° from north, or sub-parallel with St. Davids Buried Gorge. A weak conjugate set was also observed at an orientation of 177°. It should be noted that over a distance of approximately 280 m, starting at the Whirlpool-Queenston contact, only 13 of these continuous joints were observed and can be considered very widely spaced. This continuous joint set characteristically was reduced either side of the joint for approximately 50 – 100 mm and was planar, rough and had calcite infilling ranging from <1 mm to 10 mm in thickness. This joint set disappeared at greater depth as the tunnel advanced towards St. Davids Buried Gorge, suggesting that this set could be related to stress relief due to the Niagara Escarpment and Lake Ontario, and results from a strain incompatibility between the Whirlpool and Queenston. More data is necessary to establish this theory.

Underneath St. Davids Buried Gorge, another joint set perpendicular to the gorge was observed. This joint set was tight, planar, and smooth with an average spacing of 2-3 m. It was only encountered locally under the gorge and is believed to be a product of gorge erosion and stress relief.

All the structural features in the Niagara Region are widely to very widely spaced, with the exception of those under St. Davids Buried Gorge.
2.3.7 The Glacial Impact on Topographic Feature of the Niagara Region

Both the Niagara Escarpment and St. Davids Buried Gorge were present during the last glacial advance into Southern Ontario, as stated by Hough (1958) and Abidi (1992), respectively. The glacial impact on these topographic features has largely been unexplored by researchers.

The Niagara Escarpment, standing some 30 – 65 m high (Barlow, 2002), does not present a formidable topographic high for glacial advance. The ice could easily deform over top of the escarpment and in the process damage the bedrock.

The obvious area of primary damage is the lip of the escarpment, where the greatest forces would be exerted as the glacier advanced and retreated. Barlow (1999) reports that many re-entrant streams and creeks could be utilizing weak zones in the escarpment face which have been plucked by glacial action. Preliminary modelling, by the author, of glacial impacts at the escarpment indicated that failure at the face of the escarpment would only extend in from the face under wet conditions; see Figure 2.28. Under wet conditions, substantial plastic yielding is shown to occur in the subsurface, many hundreds of meters back from the escarpment face. The models are only static representations of the glacial weight, with a gap at the face. Glacial soft sediments exist at the toe of the escarpment today and were included in the modelling, as the Niagara Region was a main depositional zone during the Laurentide Ice Sheet (Menzies, 2002), which leads to the conclusion that mobile soft sediments and thin ice sheet thicknesses were present (Menzies, 2001). As the ice sheet advanced it would have deposited some of its load on the down slope side of the Escarpment and continued over top.

The ice would impinge directly on the Lockport cap rock, creating a direct hydraulic connection to the rock mass. Over time, jointing would develop and be enhanced by glacial action and the basal melt waters, potentially at high pressure. The presence of bedding planes
would also allow migration of melt waters to greater depths within the rock formation as the water enters at the face of the escarpment. The melt water could widen and deepen the existing fractures to create unstable areas, which would be removed by glacial movement or subsequent erosion of the escarpment face post-glaciation, due to creep (Barlow, 1995).

St Davids Buried Gorge has not been extensively studied. Early observations by Lyell (1845) and Spencer (1907) identified the gorge and further work was centered on the hydro power developments of the area (Barnet, 1988, Flint & Lolcama, 1985, Calkin & Brett, 1978, Karrow and Terasmae, 1970, and Hobson and Terasmae, 1969). Some of the most recent findings, during the investigations for the Niagara Tunnel Project, have revealed characteristics of the buried gorge which, taken with past findings, may suggest that St. Davids Buried Gorge was produced by tunnel valley processes.

2.3.7.1 Evidence for Tunnel Valley Formation

A new theory as to the origin of St. Davids Buried Gorge is proposed in this thesis. It is possible that the gorge was formed by tunnel valley processes, erosion underneath a glacial ice sheet by high pressure water. Many of the features which describe tunnel valleys are present in St. Davids Buried gorge, such as;

- Non-continuously falling thalweg
- Undulating longitudinal profile
- Rectilinear to slightly sinuous surface outline
- Abruptly terminating
A high point in the valley or a saddle point was utilized for the Niagara Tunnel Project location to minimize the depth of the tunnel. Drilling investigations for the project indicate that an overhang exists at depth as the borehole went from Whirlpool Sandstone to soil to Queenston. The overhang could be a relict plunge pool from a falls or it could be part of an undulating bottom which is common in tunnel valleys. In Figure 2.29, the bedrock contours indicate an...
undulating profile with deep depressions separated by a shallow reach to the north, which is how Menzies (2002) describes common characteristics of a tunnel valley. The undulating profile is also found in other high pressure systems and the melt water, trapped under the ice sheet would be under pressure. The tunnel valley bottom is often adverse to the general flow direction of the region (Hooke & Jennings, 2006) and St. Davids Buried Gorge is deeper at the Whirlpool than at the outlet at the town of St. Davids. This could be a result of erosion at the Whirlpool location subsequent to gorge formation. Spencer (1907) indicated that the rim of the gorge in the Whirlpool location has been smoothed by glacial processes indicating that ice or high pressure water forces have been acting in this area after initial gorge erosion, possibly widening and deepening the gorge. A gap, where sediment and dead ice could accumulate, at the base of the escarpment could provide a location for water to build up. With high pressure water build up, hydrofracturing could create path ways for water release at the ice sheet toe. Once a path way opens, catastrophic flows could lift slabs of rock or eroded blocks due to other joints to create a tunnel valley.

The surface outline of St. Davids Buried Gorge is characteristically rectilinear, with a shallow bend occurring at the saddle point and otherwise relatively straight stretches, which is also characteristic of tunnel valleys (Jorgensen & Sandersen, 2005).

The gorge itself terminates abruptly at the Whirlpool area, another feature of tunnel valleys. The gorge may have shared a short stretch of the Niagara River Gorge, which caused the Niagara River gorge to change its flow path when it intersected St. Davids Buried Gorge. If the tunnel valley were to have created St. Davids Buried Gorge, then the discharge would have been in a southerly direction, resulting in an out wash fan. In the area along the Niagara River Gorge the surficial soil is glaciomarine in origin and may obscure the outwash fan material.
Further investigations would be necessary to determine if in fact St. Davids Buried Gorge was originally formed by tunnel valley processes. It is apparent that the gorge was formed by water erosion over a period of time as no evidence of fault activity was observed in the Niagara Tunnel.

Figure 2.29: Bedrock elevation contours for St. Davids Buried Valley. Data used to plot the contours is taken from seismic surveys and refined with boreholes (data courtesy of Ontario Power Generation). Note the undulating profile with deep depressions to the south followed by a long shallower reach to the north. The inset location map shows the town of St. Davids and the Whirlpool location for reference.
2.3.8 Stress Evolution in Niagara Region

Throughout Southern Ontario high in situ horizontal stresses exist in the sedimentary rocks. These stresses have been locked in, due to tectonic activity during the Appalachian mountain building, from sedimentary basin effects, as well as from glacial loading and erosion.

Although there is little to no evidence of tectonic activity in the Niagara Region, the Appalachian sedimentary basin has been affected at its outer boundary by the Taconic orogeny. Stresses in the sedimentary units could be partially due to a gentle squeezing of the Appalachian sedimentary basin, without causing major deformations (Karrow and White, 2002). The weight of the overlying sediments and rock units within the basin will also generate stresses which could be locked in after glacial erosion. Menzies (2002) report glacial ice thicknesses of 500 to 1300 m would cause some stress build up and glacial rebound has been observed in glacial lake beaches throughout Southern Ontario (Stern et al., 1979).

The impact of the high horizontal stresses on engineered structures and the landscape has been summarized by Karrow and White (2002). Many projects in Southern Ontario have experienced cracking and inward movements due to the stress relief with time. It is not uncommon to have pop-ups in quarry floors and these also occurred during the construction of the Chippawa hydro power canal (Lo, 1978). The wheel pits of the Adams, Canadian Niagara (Rankin Station) and Toronto power generating station experienced inward movement of the unlined pit walls, to such an extent in the Adams station that steel I beams were bent (Karrow & White, 2002). The Thorold road tunnel which is a cut and cover tunnel passing under the Welland canal experienced cracking of the tunnel side walls shortly after construction was

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3 Paper by Perras & Diederichs 2007 included in Appendix B
completed (Bowen et al., 1976). The cracking was determined to be caused by rock squeeze. The cause of the squeeze has not been determined precisely, although it could be the result of rock creep, shale layer swelling, simply stress relaxation or a combination thereof (Bowen et al. 1976). A slot in the rock adjacent to the tunnel wall relieved the stress on the concrete liner and prevented continued crack growth (Bowen et al., 1976). Other projects in Southern Ontario have dealt with high horizontal stresses and have been reported by Aglawe & Sinclair (2002), Lieszkowszky et al. (1994), Trow & Lo (1989), Lo et al. (1979), and Lo & Morton (1975).

Horizontal to vertical stress ratios (Ko) in the order of 3 to 5 have been measured for the Niagara Tunnel Project in the Queenston Formation, with greater ratios existing in the Formations above. There can also be sharp differences in the stress state from formation to formation due to the differences in elastic properties (Haimson, 1983). The maximum horizontal stresses in the Queenston Formation range between 5.3 to 30.7 MPa and with a general trend in the ENE direction near the buried St. Davids Gorge. This topographic feature, as well as the Niagara River Gorge and the escarpment, have an influence on the stress magnitude and orientation. Locally with Niagara Falls, Canada, the stress field within the Queenston Formation rotates from 30° in the north to 60° in the south and near St. Davids Buried Gorge is due north. Haimson (2006) also reports shifts in orientation and changes in magnitude of the stress field across lithological boundaries and structural features such as faults. The range of stress measurements carried out for the Niagara Tunnel Project is presented in Table 2.4 and the data is plotted in Figure 2.30. There appears to be a jump in the stress magnitude around elevation 40 m and this could be the result of increased lateral confinement provided by Lake Ontario. This observation correlates to the potential stress relief vertical joints in the upper elevations of the Queenston Formation. The large scatter in stress measurement below elevation 40 m could be the result of
stress variations due to the topographic features or due to stress contours following the sedimentary layering, which is not perfectly horizontal.

Stresses in the rock mass are an important engineering consideration for the design of an underground excavation. Site specific measurements should be taken to ensure the most reliable results are determined for design purposes.

Figure 2.30: Stress measurements for the Niagara Tunnel Project in the Queenston Formation. (data courtesy of Ontario Power Generation)
Table 2.4: Stress measurements in the formations of the Niagara Region. Average values quoted with range in brackets. (data courtesy of Ontario Power Generation).

<table>
<thead>
<tr>
<th>Formation</th>
<th>Maximum horizontal $\sigma_H$ (MPa)</th>
<th>Minimum horizontal $\sigma_h$ (MPa)</th>
<th>Vertical $\sigma_v$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lockport</td>
<td>11.7 (11.1-12.7)</td>
<td>5.0 (1.7-8.2)</td>
<td>-</td>
</tr>
<tr>
<td>Rochester</td>
<td>8.8 (6.6-11.0)</td>
<td>1.4 (0.8-1.9)</td>
<td>0.6 (0.5-0.7)</td>
</tr>
<tr>
<td>Irondequoit</td>
<td>7.3</td>
<td>3.8</td>
<td>0.8</td>
</tr>
<tr>
<td>Reynales</td>
<td>6.0</td>
<td>3.5</td>
<td>-</td>
</tr>
<tr>
<td>Neagha</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Thorold</td>
<td>7.7 (3.4-12)</td>
<td>2.6</td>
<td>1.0</td>
</tr>
<tr>
<td>Grimsby</td>
<td>10.4 (3.4-18.3)</td>
<td>3.3 (2.4-5.3)</td>
<td>1.5 (1.2-1.8)</td>
</tr>
<tr>
<td>Power Glen</td>
<td>10.3 (8.0-11.5)</td>
<td>4.6 (2.0-6.5)</td>
<td>1.9 (1.5-2.3)</td>
</tr>
<tr>
<td>Whirlpool</td>
<td>9.6 (5.1-16.1)</td>
<td>4.0 (1.0-6.6)</td>
<td>2.2 (1.6-2.5)</td>
</tr>
<tr>
<td>Queenston</td>
<td>13.9 (5.3-30.7)</td>
<td>7.7 (1.0-18.3)</td>
<td>4.1 (2.2-5.6)</td>
</tr>
</tbody>
</table>
Characterization and classification of a site for an engineering project are fundamental steps in developing a rock support design for underground excavations. The characterization process determines the properties of the major geological units and sub-units of the project site, the layout of the topography and geology, and other related field observations. The key difference between characterization and classification is the character of the site is descriptive and the classification endeavors to create bins of similar character and anticipated behaviour. Each one feeds the other and this process is continually being refined as the project goes from a feasibility study, to preliminary design and tender, and onto a final design (Figure 3.1). The focus here is on classification and how it relates to support for TBM driven tunnels.

Figure 3.1: Characterization and classification cycle for a geological engineering project.
3.1 Rock Mass Classification Systems

Rock mass classification is used to rank the rock mass into broad categories of similar stability. The earliest classification system was proposed by Ritter, in 1879 (Hoek, 2007), who developed an empirical approach to determining support requirements. It was Terzaghi, in 1946, who captured the characteristics that dominate rock mass behaviour (Hoek, 2007) and formed the basis of further classification systems. Case histories from civil engineering were used to develop the multi-parameter classification systems such as Wickham et al.’s (1972) Rock Structure Rating (RSR) and those still used today, the Rock Tunneling Index, Q, by Barton et al. (1974), Rock Mass Rating (RMR) by Bieniawski (1973, 1989), and GSI by Hoek and Hoek et al. (1994, 1995). The Q and RMR systems however are limited to empirical design charts for rock support applications, whereas the GSI system results in values which can be directly input into numerical modeling programs for design testing. An important element of the Q and RMR systems is the Rock Quality Designation (RQD) developed by Deere et al. (1967). The RQD is defined as the percentage of intact core greater than 10 cm in the total length of the core run. The main systems, (Q, RMR, and GSI) will be discussed briefly below. The reader is referred to Hoek (2007) for further details on rock mass classification. These systems are widely used during the contracting process to determine the feasibility of the excavation process and different zones of behaviour along the length of the excavation. Many other systems have been used in the past and others continue to be developed to meet the needs of geological engineering.
3.1.1 Past Classification Systems

Many rock mass classification systems have been developed in the past and for various reasons have not been adopted for use in the engineering field. Some of the notable systems are briefly highlighted as they have contributed to the development and understanding behind those classification systems in use today.

Terzaghi, in 1946, developed a descriptive classification system which gives clear and precise definitions (Hoek 2007). It is one of the first classification systems to be used for tunnel design. Terzaghi’s descriptions (quoted directly from his paper, by Hoek (2007)) are:

- **Intact** rock contains neither joints nor hair cracks. Hence, if it breaks, it breaks across sound rock. On account of the injury to the rock due to blasting, spalls may drop off the roof several hours or days after blasting. This is known as a *spalling* condition. Hard, intact rock may also be encountered in the *popping* condition, involving the spontaneous and violent detachment of rock slabs from the sides or roof.

- **Stratified** rock consists of individual strata with little or no resistance against separation along the boundaries between the strata. The strata may or may not be weakened by transverse joints. In such rock the spalling condition is quite common.

- **Moderately jointed** rock contains joints and hair cracks, but the blocks between joints are locally grown together or so intimately interlocked that vertical walls do not require lateral support. In rocks of this type, both spalling and popping conditions may be encountered.
• *Blocky and seamy* rock consists of chemically intact or almost intact rock fragments which are entirely separated from each other and imperfectly interlocked. In such rock, vertical walls may require lateral support.

• *Crushed* but chemically intact rock has the character of crusher run. If most or all of the fragments are as small as fine sand grains and no recementation has taken place, crushed rock below the water table exhibits the properties of water-bearing sand.

• *Squeezing* rock slowly advances into the tunnel without perceptible volume increase. A prerequisite for squeezing is a high percentage of microscopic and sub-microscopic particles of micaceous minerals or clay minerals with a low swelling capacity.

• *Swelling* rock advances into the tunnel chiefly on account of expansion. The capacity to swell seems to be limited to those rocks that contain clay minerals such as montmorillonite, with a high swelling capacity.

The Rock Structure Rating (RSR) proposed by Wickham et al. (1972) was developed based on small tunnels supported by steel sets. It was one of the first systems to reference the application of shotcrete for support and it demonstrates the logic behind developing a classification system (Hoek, 2007). The system has three categories which are summed to obtain a ranking of the rock mass. The three categories are; geology, geometry and groundwater and joint condition. The geology category describes the rock types, it hardness and geological structure. The geometry category describes the joint spacing and orientation with respect to the tunnel drive direction. The ground water and joint condition category describes the joint condition and the amount of water inflow. These categories have formed the basis for the Q and RMR classification systems.
3.1.2 Rock Tunnelling Quality Index, Q

Around the same time Bieniawski proposed the RMR system, Barton et al. (1974) proposed the Rock Tunnelling Quality Index, known as the Q system. Based on a large number of case histories, the Q system incorporates rock quality, joint condition and the stress state by the following Equation (3.1)

\[
Q = \frac{RQD}{Jn} \times \frac{Jr}{Ja} \times \frac{Jw}{SRF}
\]

The factors in equation 3.1 above are described as follows and the ranges of values are in brackets;

- RQD – is the Rock Quality Designation (10-100)
- Jn – is the Joint Set Number (0.5-20)
- Jr – is the Joint Roughness Number (0.5-1.5)
- Ja – is the Joint Alteration Number (0.75-20)
- Jw – is the Joint Water Reduction Number (0.1-1)
- SRF – is the Stress Reduction Factor (0.1-20)

Each factor has a table associated with it (Barton 1974) which indexes the conditions of the factor for selection of an input value. Details of each factor can be found in Hutchinson and Diederichs (1996) or Hoek (2007).
The RQD / Jn quotient represents the rock mass structure and is a crude estimate of the block size. The block size is a key consideration and can provide a rough estimate of support spacing for surface retention (Hutchinson & Diederichs, 1996).

The second quotient is representative of the joint roughness and frictional characteristics and is weighted in favour of rough, unaltered joints in contact with each other (Hoek, 2007). When using the Q system, this factor should be determined by using the critical joint set which is most likely to cause stability issues (Hutchinson & Diederichs, 1996). If clay mineral coatings and fillings are found on the joint surfaces, this will greatly reduce the strength of the discontinuity and is extremely unfavourable for tunnel stability (Hoek, 2007).

The third and final quotient is a measure of the active stress in the rock mass and is a ratio between the “total stress” and the effective normal stress due to water pressure. Hoek (2007) states that this is a complicated empirical factor and it has proven impossible to determine an inter-block effective stress value from this quotient.

Table 3.1, as proposed by Barton (1974), is used for classification based on the evaluation of the rock mass using the determined Q value from Equation 3.1.

The Q system has also been used for support recommendations and Grimstad and Barton (1993) developed an Excavation Support Ratio (ESR) which is used to account for variable degrees of instability related to the usage of the excavation and is determined using table 3.2 (taken from Hoek, 2007). Using the ESR, span and Q values for a particular project, preliminary support recommendations can be derived using Figure 3.2.
Table 3.1: Classification of the rock mass quality base on the Q system (Barton et al., 1974)

<table>
<thead>
<tr>
<th>Q Value</th>
<th>Rock Mass Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.001 – 0.01</td>
<td>Exceptionally Poor</td>
</tr>
<tr>
<td>0.01 – 0.1</td>
<td>Extremely Poor</td>
</tr>
<tr>
<td>0.1 – 1</td>
<td>Very Poor</td>
</tr>
<tr>
<td>1 – 4</td>
<td>Poor</td>
</tr>
<tr>
<td>4 – 10</td>
<td>Fair</td>
</tr>
<tr>
<td>10 – 40</td>
<td>Good</td>
</tr>
<tr>
<td>40 – 100</td>
<td>Very Good</td>
</tr>
<tr>
<td>100 – 400</td>
<td>Extremely Good</td>
</tr>
<tr>
<td>400 -1000</td>
<td>Exceptionally Good</td>
</tr>
</tbody>
</table>

Table 3.2: Determination of ESR value for excavation support design using the empirical Q system (from Hoek, 2007).

<table>
<thead>
<tr>
<th>Excavation Category</th>
<th>ESR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Temporary mine openings.</td>
<td>3-5</td>
</tr>
<tr>
<td>Permanent mine openings, water tunnels for hydro power (excluding high pressure penstocks), pilot tunnels, drifts and headings for large excavations.</td>
<td>1.6</td>
</tr>
<tr>
<td>Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels.</td>
<td>1.3</td>
</tr>
<tr>
<td>Power stations, major road and railway tunnels, civil defense chambers, portal intersections.</td>
<td>1.0</td>
</tr>
<tr>
<td>Underground nuclear power stations, railway stations, sports and public facilities, factories.</td>
<td>0.8</td>
</tr>
</tbody>
</table>
1.3 Rock Mass Rating, RMR

Bieniawski first proposed the RMR system in 1973 for use in the design of tunnels in rock. This system has been updated by Bieniawski (1989) as more data was collected from civil engineering case histories. Six factors are ranked individually and summed using equation (3.2) to determine the overall RMR value for the particular unit of interest.

Figure 3.2: Empirical support guidelines based on the Q system. (After Grimstad and Barton, 1993, reproduced from Hoek, 2007).
\[ \text{RMR} = A_1 + A_2 + A_3 + A_4 + A_5 + B \]

(3.2)

The six factors and their ranges are as follows;

- \( A_1 \) – Uniaxial strength of the intact rock material \( (0-15) \)
- \( A_2 \) – Rock Quality Designation, RQD \( (3-20) \)
- \( A_3 \) – Spacing of discontinuities \( (5-20) \)
- \( A_4 \) – Condition of discontinuities \( (0-30) \)
- \( A_5 \) – Groundwater conditions \( (0-15) \)
- \( B \) – Orientation of discontinuities (adjustment for tunnels and mines) \( (-12 - 0) \)

The RMR classification system focuses on geometry and mechanical condition of the rock mass and lower RMR values indicate less favorable conditions for an excavation. Stresses have not been accounted for in the RMR system. The system uses a number of clearly defined and easy to use parameters, making it easy to use in the field. It is summarized in a convenient one page layout below (Figure 3.3).
### A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Range of values</th>
</tr>
</thead>
</table>
| 1. Strength of intact rock material| >10 MPa 4 - 10 MPa 2 - 4 MPa 1 - 2 MPa 0.5 - 1 MPa >0.5 MPa | For low range: Unial late compressive preferred
|                                   | Rating 16 12 7 4 2 | Uniaxial compressive preferred
| 2. Uniaxial compressive strength   | >250 MPa 100 - 250 MPa 50 - 100 MPa 25 - 50 MPa 5 - 25 MPa | Rating 8 9 8 9 8 9
| 3. Dull core Qudity (RQD)          | 0% - 10% 25% - 50% 50% - 75% 75% - 90% | Rating 8 9 8 9
| 4. Spacing of discontinuities      | > 2 m 0.6 - 2 m 200 - 600 m 600 - 2000 m | Rating 8 9 8 9
| 5. Condition of discontinuities   | Very rough surfaces  Slightly rough surfaces  Slightly weathered walls  Moderately weathered walls  Highly weathered walls  Soft gouge >5 mm thick or Separation > 5 mm Continuous | Rating 8 9 8 9 8 9
|                                   | Rating 30 25 20 19 9 | Unfavorable
| 6. Groundwater                     | Inflow per 10 m tunnel length (m²) 3/1 (Mill water press 1.5) (Major principal o)  | Rating 8 9 8 9 8 9
|                                   | None 0.1 0.2 0.5 1.0 1.5 2.0 | Unfavorable
|                                   | General conditions  Complexly dry  Damp  Wet  Chipping  Flowing | Rating 8 9 8 9 8 9

### B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)

<table>
<thead>
<tr>
<th>Strike and dip orientations</th>
<th>Very Favorable</th>
<th>Favorable</th>
<th>Fair</th>
<th>Unfavorable</th>
<th>Very Unfavorable</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tunnels &amp; mines</td>
<td>0</td>
<td>-2</td>
<td>-5</td>
<td>-10</td>
<td>-12</td>
</tr>
<tr>
<td>Foundations</td>
<td>0</td>
<td>-2</td>
<td>-7</td>
<td>-15</td>
<td>-25</td>
</tr>
<tr>
<td>Slopes</td>
<td>0</td>
<td>-5</td>
<td>-25</td>
<td>-50</td>
<td></td>
</tr>
</tbody>
</table>

### C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

<table>
<thead>
<tr>
<th>Class number</th>
<th>Description</th>
<th>I</th>
<th>II</th>
<th>III</th>
<th>IV</th>
<th>V</th>
</tr>
</thead>
<tbody>
<tr>
<td>100 - 61</td>
<td>Very good rock</td>
<td>Good rock</td>
<td>Fair rock</td>
<td>Poor rock</td>
<td>Very poor rock</td>
<td></td>
</tr>
<tr>
<td>59 - 61</td>
<td>Moderate rock</td>
<td>Good rock</td>
<td>Fair rock</td>
<td>Poor rock</td>
<td>Very poor rock</td>
<td></td>
</tr>
<tr>
<td>45 - 41</td>
<td>Fair rock</td>
<td>Good rock</td>
<td>Fair rock</td>
<td>Poor rock</td>
<td>Very poor rock</td>
<td></td>
</tr>
<tr>
<td>25 - 45</td>
<td>Poor rock</td>
<td>Fair rock</td>
<td>Poor rock</td>
<td>Very poor rock</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### D. BRAMINING OF ROCK CLASS Ranges

<table>
<thead>
<tr>
<th>Average stand-up time</th>
<th>Cohesion of rock mass (MPa)</th>
<th>Friction angle of rock mass (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20 yrs for 15 m span</td>
<td>&gt; 400</td>
<td>&gt; 45</td>
</tr>
<tr>
<td>1 year for 10 m span</td>
<td>300 - 400</td>
<td>25 - 45</td>
</tr>
<tr>
<td>1 week for 5 m span</td>
<td>200 - 300</td>
<td>15 - 25</td>
</tr>
<tr>
<td>19 hours for 2.5 m span</td>
<td>100 - 200</td>
<td>10 - 15</td>
</tr>
<tr>
<td>30 mins for 1 m span</td>
<td>&lt; 100</td>
<td>&lt; 10</td>
</tr>
</tbody>
</table>

### E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY conditions

<table>
<thead>
<tr>
<th>Discontinuity length (peristance)</th>
<th>Rating</th>
<th>Discontinuity separation (aperture)</th>
<th>Rating</th>
<th>Roughness</th>
<th>Rating</th>
<th>Instilling (gouge)</th>
<th>Rating</th>
<th>Weathering</th>
<th>Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 1 m</td>
<td>6</td>
<td>&lt; 0.1 mm</td>
<td>6</td>
<td>Very rough</td>
<td>6</td>
<td>Unweathered</td>
<td>6</td>
<td>1</td>
<td>6</td>
</tr>
<tr>
<td>1 - 3 m</td>
<td>4</td>
<td>0.1 - 1.9 mm</td>
<td>4</td>
<td>Rough</td>
<td>4</td>
<td>Hard filling &lt; 5 mm</td>
<td>4</td>
<td>2</td>
<td>4</td>
</tr>
<tr>
<td>3 - 10 m</td>
<td>2</td>
<td>1 - 5 mm</td>
<td>2</td>
<td>Slightly rough</td>
<td>2</td>
<td>Hard filling &lt; 5 mm</td>
<td>4</td>
<td>2</td>
<td>4</td>
</tr>
<tr>
<td>10 - 20 m</td>
<td>1</td>
<td>&gt; 5 mm</td>
<td>1</td>
<td>Smooth</td>
<td>1</td>
<td>Soft filling &lt; 5 mm</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>&gt; 20 m</td>
<td>9</td>
<td></td>
<td>9</td>
<td>Slickensided</td>
<td>9</td>
<td>Soft filling &gt; 5 mm</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
</tbody>
</table>

### F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELLING

<table>
<thead>
<tr>
<th>Strike parallel to tunnel axis</th>
<th>Strike perpendicular to tunnel axis</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drive with dip - Dip 45 - 90°*</td>
<td>Drive with dip - Dip 20 - 45°*</td>
</tr>
<tr>
<td>Drive against dip - Dip 20 - 45°*</td>
<td>Drive against dip - Dip 45 - 90°*</td>
</tr>
<tr>
<td>Very Favorable</td>
<td>Favorable</td>
</tr>
<tr>
<td>Fair</td>
<td>Unfavorable</td>
</tr>
</tbody>
</table>

* Some conditions are mutually exclusive. For example, if instilling is present, the roughness of the surface will be overprinted by the influence of the gouge. In such cases use A.4 directly.

** Modified after Wickham et al (1972).

Figure 3.3: Rock Mass Rating (RMR) system after Bieniawski (1989).
Table 3.3: Rockmass classification system rankings for RMR suggested by Bieniawski (1993).

<table>
<thead>
<tr>
<th>Rock Mass Class</th>
<th>Description</th>
<th>RMR Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Very Good Rock</td>
<td>81 - 100</td>
</tr>
<tr>
<td>II</td>
<td>Good Rock</td>
<td>61 – 80</td>
</tr>
<tr>
<td>III</td>
<td>Fair Rock</td>
<td>41 – 60</td>
</tr>
<tr>
<td>IV</td>
<td>Poor Rock</td>
<td>21 – 40</td>
</tr>
<tr>
<td>V</td>
<td>Very Poor Rock</td>
<td>0 - 21</td>
</tr>
</tbody>
</table>

Bieniawski suggested that Table 3.3 be used in conjunction with the rockmass classification system based on the relationship above and for the output of the RMR system to rank the rock mass.

The RMR system has been correlated to rock support installed on many civil engineering projects to develop empirical rock support requirements. As the case histories used to develop these recommendations are from civil engineering projects, they relate to excavations at shallow depth. The support guidelines offered by Bieniawski (1993) are presented in Table 3.4 and are based on a 10 m wide horseshoe shaped, drill and blast tunnel under 25 MPa of vertical stress. Other authors have adapted and modified the RMR system for specific applications and the reader is referred to Hutchinson and Diederichs (1996) for further information in this regard.
Table 3.4: Support recommendations based on RMR rating (reproduced from Hoek 2007 after Bieniawski 1993).

<table>
<thead>
<tr>
<th>Rock mass class</th>
<th>Excavation</th>
<th>Rock bolts (20 mm diameter, fully grouted)</th>
<th>Shotcrete</th>
<th>Steel sets</th>
</tr>
</thead>
<tbody>
<tr>
<td>I - Very good rock</td>
<td>Full face, 3 m advance.</td>
<td>Generally no support required except spot bolting.</td>
<td>Locally, bolts in crown 3 m long, spaced 2.5 m with occasional wire mesh.</td>
<td>None.</td>
</tr>
<tr>
<td>RMR: 81-100</td>
<td></td>
<td></td>
<td>50 mm in crown where required.</td>
<td>None.</td>
</tr>
<tr>
<td>II - Good rock</td>
<td>Full face, 1-1.5 m advance. Complete support 20 m from face.</td>
<td></td>
<td>Systematic bolts 4 m long, spaced 1.5 - 2 m in crown and walls with wire mesh in crown.</td>
<td></td>
</tr>
<tr>
<td>RMR: 51-60</td>
<td></td>
<td></td>
<td>50-100 mm in crown and 30 mm in sides.</td>
<td>None.</td>
</tr>
<tr>
<td>III - Fair rock</td>
<td>Top heading and bench 1.5-3 m advance in top heading. Commence support after each blast. Complete support 10 m from face.</td>
<td>Systematic bolts 4-5 m long, spaced 1-1.5 m in crown and walls with wire mesh.</td>
<td>100-150 mm in crown and 100 mm in sides.</td>
<td>Light to medium ribs spaced 1.5 m where required.</td>
</tr>
<tr>
<td>RMR: 41-60</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>IV - Poor rock</td>
<td>Top heading and bench 1.0-1.5 m advance in top heading. Install support concurrently with excavation, 10 m from face.</td>
<td>Systematic bolts 4-5 m long, spaced 1-1.5 m in crown and walls with wire mesh.</td>
<td>100-150 mm in crown and 100 mm in sides.</td>
<td>Light to medium ribs spaced 1.5 m where required.</td>
</tr>
<tr>
<td>RMR: 21-40</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>V - Very poor rock</td>
<td>Multiple drifts 0.5-1.5 m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting.</td>
<td>Systematic bolts 5-6 m long, spaced 1-1.5 m in crown and walls with wire mesh. Bolt invert.</td>
<td>150-200 mm in crown, 150 mm in sides, and 50 mm on face.</td>
<td>Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required. Close invert.</td>
</tr>
<tr>
<td>RMR: &lt; 20</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### 3.1.4 Geological Strength Index, GSI

The Geological Strength Index (GSI) system was first proposed by Hoek in 1994 to meet the need of a classification system which could be used on weak rock masses and to link the Hoek-Brown (Hoek et al., 2002) constants m and s to measurement or observations from the field. The Hoek-Brown constants (Hoek et al., 2002) were initially linked to Bieniawski’s RMR classification system, however the lowest possible RMR value is 8 and this system does not accurately represent jointed rock masses. To overcome this problem and to minimize confusion of calculating GSI from Q or RMR, Hoek proposed the following table to determine the GSI value based on observed structure and joint surface conditions (Figure 3.4).
**Figure 3.4: Original GSI chart proposed by Hoek (1994)**

The Generalised Hoek-Brown Criterion is given by:

\[
\sigma_1' = \sigma_3' + \sigma_c \left( \frac{m_b \sigma_3'}{\sigma_c} + s \right)^a
\]

- \(\sigma_1'\) = major principal effective stress at failure
- \(\sigma_3'\) = minor principal effective stress at failure
- \(\sigma_c\) = uniaxial compressive strength of intact pieces of rock
- \(m_b\), \(s\) and \(a\) are constants which depend on the composition, structure and surface conditions of the rock mass.

The table below provides the values of various parameters for different surface conditions and rock structures.

<table>
<thead>
<tr>
<th>STRUCTURE</th>
<th>SURFACE CONDITION</th>
<th>(m_{Y/m_i})</th>
<th>(a)</th>
<th>(E_m)</th>
<th>(\nu)</th>
<th>(GSI)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BLOCKY - very well interlocked, undisturbed rock mass consisting of cubical blocks formed by three orthogonal discontinuity sets</td>
<td>VERY GOOD, very rough, unweathered surfaces</td>
<td>0.60</td>
<td>0.190</td>
<td>0.5</td>
<td>75.000</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td>GOOD, slightly weathered, iron stained surfaces</td>
<td>0.40</td>
<td>0.062</td>
<td>0.5</td>
<td>40.000</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td>FAIR, moderately weathered or altered surfaces</td>
<td>0.26</td>
<td>0.015</td>
<td>0.5</td>
<td>20.000</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td>POOR, highly weathered surfaces with compact coatings or fillings containing angular fragments</td>
<td>0.16</td>
<td>0.003</td>
<td>0.5</td>
<td>9.000</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>VERY POOR, highly weathered surfaces with soft clay coatings or fillings</td>
<td>0.08</td>
<td>0.0004</td>
<td>0.5</td>
<td>3.000</td>
<td>0.25</td>
</tr>
<tr>
<td>VERY BLOCKY - interlocked, partially disturbed rock mass with multifaceted angular blocks formed by four or more discontinuity sets</td>
<td>VERY GOOD, very rough, unweathered surfaces</td>
<td>0.40</td>
<td>0.062</td>
<td>0.5</td>
<td>40.000</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td>GOOD, slightly weathered, iron stained surfaces</td>
<td>0.29</td>
<td>0.021</td>
<td>0.5</td>
<td>24.000</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>FAIR, moderately weathered or altered surfaces</td>
<td>0.16</td>
<td>0.003</td>
<td>0.5</td>
<td>9.000</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>POOR, highly weathered surfaces with compact coatings or fillings containing angular fragments</td>
<td>0.11</td>
<td>0.001</td>
<td>0.5</td>
<td>5.000</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>VERY POOR, highly weathered surfaces with soft clay coatings or fillings</td>
<td>0.07</td>
<td>0.0004</td>
<td>0.5</td>
<td>2.500</td>
<td>0.3</td>
</tr>
<tr>
<td>BLOCKY/SEAMY - folded and faulted with many intersecting discontinuities forming angular blocks</td>
<td>VERY GOOD, very rough, unweathered surfaces</td>
<td>0.24</td>
<td>0.012</td>
<td>0.5</td>
<td>18.000</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>GOOD, slightly weathered, iron stained surfaces</td>
<td>0.17</td>
<td>0.004</td>
<td>0.5</td>
<td>10.000</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>FAIR, moderately weathered or altered surfaces</td>
<td>0.12</td>
<td>0.001</td>
<td>0.5</td>
<td>6.000</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>POOR, highly weathered surfaces with compact coatings or fillings containing angular fragments</td>
<td>0.08</td>
<td>0.0001</td>
<td>0.5</td>
<td>3.000</td>
<td>0.3</td>
</tr>
<tr>
<td></td>
<td>VERY POOR, highly weathered surfaces with soft clay coatings or fillings</td>
<td>0.06</td>
<td>0.00001</td>
<td>0.5</td>
<td>2.000</td>
<td>0.3</td>
</tr>
<tr>
<td>CRUSHED - poorly interlocked, heavily broken rock mass with a mixture of angular and rounded blocks</td>
<td>VERY GOOD, very rough, unweathered surfaces</td>
<td>0.17</td>
<td>0.004</td>
<td>0.5</td>
<td>10.000</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>GOOD, slightly weathered, iron stained surfaces</td>
<td>0.12</td>
<td>0.001</td>
<td>0.5</td>
<td>6.000</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>FAIR, moderately weathered or altered surfaces</td>
<td>0.08</td>
<td>0.0001</td>
<td>0.5</td>
<td>3.000</td>
<td>0.3</td>
</tr>
<tr>
<td></td>
<td>POOR, highly weathered surfaces with compact coatings or fillings containing angular fragments</td>
<td>0.06</td>
<td>0.00001</td>
<td>0.5</td>
<td>2.000</td>
<td>0.3</td>
</tr>
<tr>
<td></td>
<td>VERY POOR, highly weathered surfaces with soft clay coatings or fillings</td>
<td>0.04</td>
<td>0.000001</td>
<td>0.5</td>
<td>1.000</td>
<td>0.3</td>
</tr>
</tbody>
</table>
In 2005, Hoek et al. recognized the need to further develop a system which could be used for layered and undisturbed rock masses, such as molasse. A molasse is a sedimentary rock which formed after the main orogenesis, as the mountain range was eroded. A molasse can be comprised of sandstones, siltstones, mudstones or marls and conglomerates, and restricted limestone horizons can also occur (Hoek et al., 2005). They are generally flat lying, with dips greater than 30° occurring only locally. Structures, such as gravity faults and late orogenic over thrusts can occur, but generally only affect the rock mass locally.

Underground, the confinement and constant environmental influences allow molasse rock masses to be well preserved at depth and their appearance is very different than the same rock mass exposed at surface where weathering can occur and quickly deteriorate the rock mass. For underground structures and evaluating rock mass from fresh core, the following GSI chart (Figure 3.5) was proposed for molasse type rock masses under confinement by Hoek et al. (2005). Another GSI chart was produced for molasse at surface, which is not presented here, but can be found in Hoek et al. (2005). Since the bedding planes, at depth, are almost indistinct and are generally intact (not pre-split) then the GSI value should remain high and the bedding planes will be accounted for in the strength and modulus of the material (Hoek et al., 2005). This approach generates isotropic rock mass properties, either using Roclab (from RocScience) or Hoek-Brown constants (Hoek et al., 2002). The resulting rock mass properties can be implemented directly into numerical modeling software programs.
Figure 3.5: GSI chart for use with confined molasse type rock masses. M1 at depth and M2 at surface (Hoek et al., 2004).
The input values to generate the rock mass properties are; UCS, $E_i$, $m_i$, and GSI. The first three are determined by laboratory testing and the GSI value is determined from observations in the field and core logging. These values are input into Roclab (from RocScience) or the equations proposed by Hoek et al. (2002) to generate the $m_b$, $s$, and $a$ values for the Hoek-Brown parameters or the Mohr-Coulomb values cohesion, friction and tension. These values can be used for rock masses which are close to being homogeneous and isotropic. If the rock mass is anisotropic however, the numerical modeling software cannot simulate the mechanistic behaviour correctly. This will be discussed further in Chapters 4 and 5.

The GSI system is becoming more commonly used for construction purposes throughout the world. It is easily implemented and parameters needed for numerical modeling are directly determined from the system. With increasing computational power, numerical modeling software is becoming more powerful and faster. This has increased the use of numerical modeling for design purposes.

### 3.2 Rock mass properties of the Niagara Stratigraphy

The geology of the Niagara Stratigraphy, as discussed previously, varies from limestone, shale, sandstone and siltstone and the rock mass properties vary considerably. For example, the unconfined compressive strength (UCS) of the Neagha shale is in the order of 18 MPa and that of the Whirlpool sandstone is 180 MPa. Even within one formation, distinct shale partings, up to 200 mm in thickness, such as in the Grimsby, represent a weak layer within a stronger layer. The weak interbeds of shale will control the failure mechanism, which will be dampened by the stronger unit.
To conduct numerical modeling the input parameters which must be determined are:

- GSI / RMR / Q
- Hoek-Brown constant, \( m_i \)
- UCS
- Poisson’s ratio, \( \nu \)
- Young’s modulus, \( E_{\text{intact}} \)

The first parameter is based on field and core observations for different zones along the tunnel alignment. The remaining parameters are based on laboratory testing of intact pieces of core. Hoek (1994) defines intact rock as the material in between structural discontinuities.

For the Niagara Tunnel Project, the various rock mass properties are listed in Table 3.5 which is modified from Perras & Diederichs (2007) to include Poisson’s ratio, \( \nu \). These properties are necessary for numerical modeling and were determined over several years through investigation programs for the Niagara Tunnel Project.

The rock mass properties of the formation above the Queenston display behaviour consistent with similar rock types from other locations. The stiff materials are generally brittle and almost complete strain recovery is possible within the elastic range. The samples tested fall between medium to high strength, as shown in Figure 3.6.
Table 3.5: Rock mass properties of the Niagara Stratigraphy (modified from Perras & Diederichs, 2007, original data courtesy of Ontario Power Generation)

<table>
<thead>
<tr>
<th>Formation</th>
<th>RMR/GSI</th>
<th>m_i</th>
<th>UCS (MPa)</th>
<th>E_i (GPa)</th>
<th>ν</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lockport</td>
<td>70/80</td>
<td>9</td>
<td>151</td>
<td>67</td>
<td>0.37</td>
</tr>
<tr>
<td>Decew</td>
<td>69/70</td>
<td>9</td>
<td>128</td>
<td>51</td>
<td>0.30</td>
</tr>
<tr>
<td>Rochester</td>
<td>64/77</td>
<td>7</td>
<td>41</td>
<td>11</td>
<td>0.30</td>
</tr>
<tr>
<td>Irondequoit</td>
<td>72/82</td>
<td>10</td>
<td>89</td>
<td>60</td>
<td>0.32</td>
</tr>
<tr>
<td>Reynales</td>
<td>67/77</td>
<td>9</td>
<td>101</td>
<td>33</td>
<td>0.25</td>
</tr>
<tr>
<td>Neagha</td>
<td>56/66</td>
<td>6</td>
<td>18</td>
<td>4</td>
<td>0.45</td>
</tr>
<tr>
<td>Thorold</td>
<td>78/83</td>
<td>18</td>
<td>129</td>
<td>53</td>
<td>0.22</td>
</tr>
<tr>
<td>Grimsby Sandstone</td>
<td>70/75</td>
<td>17</td>
<td>146</td>
<td>43</td>
<td>0.24</td>
</tr>
<tr>
<td>Grimsby Shale</td>
<td>60/60</td>
<td>7</td>
<td>35</td>
<td>8</td>
<td>0.25</td>
</tr>
<tr>
<td>Power Glen Sandstone</td>
<td>63/68</td>
<td>17</td>
<td>152</td>
<td>59</td>
<td>0.21</td>
</tr>
<tr>
<td>Power Glen Shale</td>
<td>60/60</td>
<td>7</td>
<td>22</td>
<td>9</td>
<td>0.42</td>
</tr>
<tr>
<td>Whirlpool</td>
<td>85/87</td>
<td>19</td>
<td>180</td>
<td>50</td>
<td>0.19</td>
</tr>
<tr>
<td>Queenston</td>
<td>65</td>
<td>8</td>
<td>46</td>
<td>16</td>
<td>0.35</td>
</tr>
</tbody>
</table>
Figure 3.6: Compressive strength to intact modulus comparison for test samples for the Niagara Tunnel Project (image courtesy of Ontario Power Generation).
Strength variability within the sandstone units is due to shale interbedding and variations in mineral content. For the most part these variations, within the competent sandstone units, are of little consequence to the stability of the excavation due to the thin nature of the shale interbeds. However, the interface between the shale and sandstone is a center of nucleation for stress induced fracture propagation, as well as delamination locally near vertical structure, which will be discussed later in this section.

The Queenston siltstone has roughly 500 microstrain units which are non-recoverable during loading and unloading cycles, as shown in Figure 3.7. It is a brittle material with little curvature exhibited in UCS stress-strain plots post peak and abrupt termination at sample failure. This is classic class II failure as described in the ISRM suggested methods (Fairhurst & Hudson, 1999).

Measuring the degree of anisotropy has been proposed by several authors based on UCS, (Ramamurthy, 1993), point load index (Tsidzki, 1990) and transversly isotropic elastic constants (Kwasniewski, 1984).
Figure 3.7: Typical unconfined compressive strength test on core from the Queenston Formation (image courtesy of Ontario Power Generation).

Table 3.6: Classification based on the anisotropy ratio (quoted from Colak & Unlu (2004) after Ramamurthy (1993)).

<table>
<thead>
<tr>
<th>Range</th>
<th>Class</th>
</tr>
</thead>
<tbody>
<tr>
<td>$1.0 &lt; R_c &lt; 1.1$</td>
<td>Isotropic</td>
</tr>
<tr>
<td>$1.1 &lt; R_c &lt; 2.0$</td>
<td>Low anisotropy</td>
</tr>
<tr>
<td>$2.0 &lt; R_c &lt; 4.0$</td>
<td>Medium anisotropy</td>
</tr>
<tr>
<td>$4.0 &lt; R_c &lt; 6.0$</td>
<td>High anisotropy</td>
</tr>
<tr>
<td>$6.0 &lt; R_c$</td>
<td>Very high anisotropy</td>
</tr>
</tbody>
</table>


Ramamurthy (1993) suggested using UCS values to determine an anisotropy ratio defined as:

\[ R_c = \frac{\sigma_{ci(90)}}{\sigma_{ci(min)}} \]  

(3.3)

where \( \sigma_{ci(90)} \) is the uniaxial compressive strength of a sample tested with the plane of anisotropy perpendicular to the loading direction and \( \sigma_{ci(min)} \) is the minimum strength of oriented core samples. Ramamurthy (1993) suggests Table 3.6 to classify the degree of anisotropy based on the anisotropy ratio.

Tsidzi (1990) proposed a point load strength anisotropy index \( I_{a(50)} \) and defined it as:

\[ I_{a(50)} = \frac{I_{s(50)\perp}}{I_{s(50)\parallel}} \]  

(3.4)

where \( I_{a(50)\perp} \) and \( I_{a(50)\parallel} \) are the standard point load values perpendicular and parallel to the plane of anisotropy, respectively. A classification system, which was proposed by Colak and Unlu (2004), for the point load index is presented below in Table 3.7.

Using the transversely isotropic elastic constants, \( E_1, E_2, G_2, v_1 \) and \( v_2 \), Leknitskii (reported by Kwasniewski, 1984) proposed the equations 3.5, 3.6 and 3.7 for the classification proposed in Table 3.8.
\[ n = (2k + m)^{0.5} \]  

\[ k = \sqrt{\frac{E_1 - \nu_2^2}{E_2}} \]  

\[ m = \sqrt{\frac{E_1 - 2\nu_2(1 + \nu_1)}{G_2}} \]  

Table 3.7: Classification of anisotropic rocks based on the point load strength anisotropy index \((I_{a(50)})\) (quoted from Colak & Unlu, 2004).

<table>
<thead>
<tr>
<th>Nature of rock</th>
<th>Range</th>
<th>Class</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very weakly foliated or non foliated</td>
<td>(I_{a(50)} &lt; 1.1)</td>
<td>Quasi-isotropic</td>
</tr>
<tr>
<td>Weakly foliated</td>
<td>(1.1 &lt; I_{a(50)} &lt; 1.5)</td>
<td>Fairly anisotropic</td>
</tr>
<tr>
<td>Moderately foliated</td>
<td>(1.5 I_{a(50)} &lt; 2.5)</td>
<td>Moderately anisotropic</td>
</tr>
<tr>
<td>Strongly foliated</td>
<td>(2.5 I_{a(50)} &lt; 3.5)</td>
<td>Highly anisotropic</td>
</tr>
<tr>
<td>Very strongly foliated</td>
<td>(3.5 &lt; I_{a(50)})</td>
<td>Very highly anisotropic</td>
</tr>
</tbody>
</table>

Table 3.8: Classification of transversely isotropic materials on the basis of elastic anisotropy parameters (quoted from Colak & Unlu, 2004).

<table>
<thead>
<tr>
<th>Range</th>
<th>Class</th>
</tr>
</thead>
<tbody>
<tr>
<td>(n &lt; 2.1)</td>
<td>Semi isotropic</td>
</tr>
<tr>
<td>(2.1 &lt; n &lt; 2.5)</td>
<td>Low anisotropy</td>
</tr>
<tr>
<td>(2.5 &lt; n &lt; 3.0)</td>
<td>Medium anisotropy</td>
</tr>
<tr>
<td>(3.0 &lt; n)</td>
<td>High anisotropy</td>
</tr>
</tbody>
</table>
Figure 3.8: Relationship between depth and unconfined compressive strength data from measurements on the Queenston Formation (data courtesy of Ontario Power Generation).

Based on the above mentioned anisotropy classification systems, the Queenston Formation was analyzed to determine the degree of anisotropy using the Anisotropy Ratio ($R_e$). The UCS data is plotted in Figure 3.8 against elevation since anisotropy is stress dependent (Chappel, 1989) and a wide range of values is seen at elevations below 0 m. The wide range of values is due to the variability in the composition of the samples.
The shale content of the Queenston Formation can be extremely variable from one elevation to another. This is due to the complex compound deltaic depositional environment.

Figure 3.9: Relationship between shale content and compressive strength for the Queenston Formation (image courtesy of Ontario Power Generation).
Figure 3.10: Relationship between shale content and Young's modulus for the Queenston Formation (image courtesy of Ontario Power Generation).
Table 3.9: Anisotropy ratios and classification for the Queenston Formation (based on Ramamurthy, 1993).

<table>
<thead>
<tr>
<th></th>
<th>$\sigma_{ci(90)}$ (MPa)</th>
<th>$\sigma_{ci(min)}$ (MPa)</th>
<th>$R_c$</th>
<th>Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper Elevation</td>
<td>39.1</td>
<td>11.8</td>
<td>3.3</td>
<td>medium</td>
</tr>
<tr>
<td>All Data</td>
<td>72.5</td>
<td>11.8</td>
<td>6.1</td>
<td>high</td>
</tr>
</tbody>
</table>

The shale content has been shown to have an inverse affect on the compressive strength and Young’s modulus as shown in Figure 3.9 and Figure 3.10, respectively. Due to the wide range of values for the UCS, the anisotropy ratio ($R_c$) was calculated using the upper data set (> 0 m elevation) and all the data. The results of the $R_c$ calculation are present in Table 3.9 and the Queenston anisotropy ranges from medium to high based on Ramamurthy’s (1993) classification.

3.2.1 Swelling Mechanisms in Rock

Many projects have been constructed on and in swelling ground. In Ottawa, Ontario, many foundations are constructed in the black shale of the Billings Formation, which exhibits swelling behavior, and can lead to foundation cracking and heave if not properly considered in the design phase (Harper et al., 1979). Tunnels in Europe have been constructed through anhydritic shale rocks, (Steiner, 1993), which can swell when water transforms the anhydrate into gypsum. This swelling process results in extreme heave and crushing of reinforced invert arches (Steiner 1993). Research is being conducted into underground cavern storage of nuclear waste in swelling shale formations, so that the swelling nature can be used to seal fractures generated during construction. One such facility will be the NAGRA nuclear waste site in Switzerland, where the repository will
be located in the Opalinuston, which is German for Opalinus Clay Shale. In the Niagara Region, the shale units all exhibit swelling potential to varying degrees. Of these units, the Queenston Formation has the highest swelling potential. The current Niagara Tunnel Project is being excavated by a 14.4 m tunnel boring machine (TBM) in Niagara Falls, Ontario. This project will pass through the Queenston Formation and an impervious liner is being installed to inhibit the swelling process.

Several swelling mechanisms of rocks, in general, have been reported by researchers and include pyrite oxidation, hydration of anhydrite, double layer theory, and ionic diffusion. The first three are described briefly below for completeness and the fourth is described in more detail as it pertains to the Queenston Formation in which the Niagara Tunnel Project is being constructed.

The black shale of the Billings Formation in and around Ottawa has caused basement heave in buildings of this region (Harper et al., 1979). The heave results from a series of chemical reactions in which carbonates react with sulfuric acid formed from oxidation of pyrite, catalyzed by bacteria. This reaction forms crystals of jarosite (2KFe₃(SO₄)₂(OH)₆) and gypsum (CaSO₄ . 2H₂O), often in existing fractures and bedding plane partings (Harper et al., 1979). The new crystals cause expansion of the discontinuities.

The transformation of anhydrite, by hydration, into gypsum causes a volume increase and results in swelling of the rock mass. It was found, by Madsen and Nuesch (1991) that a mixture of 70-75 % anhydrite and 10-25 % clay results in the maximum swelling potential. The clay acts as a catalyst for the transformation of the anhydrite and a water source which is needed for the

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4 Information from this sub-section was presented at cYEGC conference (Perras & Diederichs 2007)
reaction (Madsen & Nuesch, 1991). It has been noted that this process can result in a volume increase as much as 60% and pressures of 6-7 MPa (Madsen & Nuesch, 1991), have been noted at the Adler Tunnel.

The swelling behaviour of clay minerals has been described using the double layer theory. The theory, which Olson and Mesri (1970) agreed accounts well for the swelling behaviour of clay minerals, implies that clay minerals are represented by two thin silica rich plates that are held together by aluminum or other metallic molecules and water. The silica sheets acquire a negative charge when metallic minerals are substituted into the layer and this negative charge is balanced by the positively charged aluminum layer (Friend & Hunter, 1970). When a source of water is encountered, water molecules can be absorbed between the plates, causing expansion as illustrated in Figure 3.11. Some clay minerals such as kaolinite or montmorillonite can absorb large volumes of water.

![Figure 3.11: Clay swelling mechanism.](image)
Ionic diffusion is the process of transferring ions from the inner pore space of the rock mass to an ambient fluid (i.e. in a tunnel or reservoir) which is not in chemical equilibrium with the pore fluid. The change in chemistry of the pore fluid causes the pore volume to change. As diffusion continues, the rock mass begins to swell if there is an opening for stress relief, such as an excavation.

Swelling of the shale units in the Niagara Region is well documented, and is reported in terms of ‘swell potential’, or the strain rate per log cycle of time in days (Lee & Lo, 1989). Significant time dependent problems have occurred in the upper geological units of the Niagara Region, namely the shaley layer of the Gasport Member in the Lockport Formation (Bowen, 1976). The Canadian Niagara Power Company experienced problems in the wheel pit at the Toronto Power Station and monitoring of the closure was recorded as early as 1905 (Bowen et al., 1976). Similar problems occurred at the Thorold Tunnel, where significant horizontal cracking was observed in the portal areas (Bowen et al., 1976). Of the geological units in the Niagara Region, the Queenston Formation exhibits the highest swelling potential and for this reason the current Niagara Tunnel Project is installing a double layer membrane to eliminate the possibility of swelling through the diffusion process, by limiting the interaction of the ground water and fresh water of the tunnel.
X-ray defraction results shown in Table 3.10 agree with this explanation of the swelling mechanism as there are no salt minerals identified in the swollen samples and halite is present in the fresh samples. The swollen samples should have, as found, a low concentration of salt ions, if the ionic diffusion swelling mechanism holds, as these ions have been diffused from the sample during the swelling process. Also only trace amounts of swelling clay minerals were identified and considered to be of such a low percentage as to have an insignificant contribution to the swelling mechanism through double clay layer water absorption. A detailed swelling investigation report was conducted as part of this research and is presented in Appendix A.
Table 3.10: XRD sample results showing the most abundant minerals, the percentage is based on the 100% peak for the listed minerals only. Data from report in Appendix A and samples as follows; Old – Queenston core from 1992, Sealed – Queenston core preserved from 1992, S2 swell – Queenston core, from 2007, after 100 days of free swell testing, S3a fresh & S3b fresh – Queenston core, from 2007, preserved until XRD testing and S3a swell – Queenston core, from 2007, after 100 days of free swell testing. Note that all S3 samples are taken from within 30 cm of each other.

<table>
<thead>
<tr>
<th>Mineral</th>
<th>Old</th>
<th>Sealed</th>
<th>S2 swell</th>
<th>S3a fresh</th>
<th>S3b fresh</th>
<th>S3a swell</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quartz</td>
<td>32</td>
<td>45</td>
<td>31</td>
<td>35</td>
<td>47</td>
<td>27</td>
</tr>
<tr>
<td>Clinochlore</td>
<td>16</td>
<td>13</td>
<td>19</td>
<td>18</td>
<td>15</td>
<td>18</td>
</tr>
<tr>
<td>Muscovite</td>
<td>35</td>
<td>37</td>
<td>49</td>
<td>41</td>
<td>28</td>
<td>49</td>
</tr>
<tr>
<td>Calcite</td>
<td>5</td>
<td>3</td>
<td>-</td>
<td>1</td>
<td>4</td>
<td>3</td>
</tr>
<tr>
<td>Halite</td>
<td>10</td>
<td>-</td>
<td>-</td>
<td>3</td>
<td>3</td>
<td>-</td>
</tr>
<tr>
<td>Other</td>
<td>2</td>
<td>2</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>Montmorillonite</td>
<td>-</td>
<td>-</td>
<td>Trace</td>
<td>-</td>
<td>Trace</td>
<td>-</td>
</tr>
<tr>
<td>Illite</td>
<td>-</td>
<td>Trace</td>
<td>-</td>
<td>-</td>
<td>Trace</td>
<td>-</td>
</tr>
</tbody>
</table>

The swelling process, in the Queenston, is associated with ionic diffusion of salts from the connate pore water in the rock, and a corresponding reduction in capillary and surface tension on the minerals around the pore (Hawlader et al., 2003). The fresh water creates a diffusion gradient and salt ions are then transported from the pore space into the fresh water source. The removal of the salt ions from the pore space relieves the attractive forces of the surrounding molecules and causes expansion as illustrated in Figure 3.13.
Salt ions (●) creating stable rock pore

Pore water dilution results in swell

Figure 3.13: Illustration of the ion diffusion swelling mechanism, representative of the Queenston Formation in southern Ontario, where salt ions create inward attractive forces in the pore space and when diffused into the ground water, cause swelling due to the removal of the attractive forces.

The mechanism for swelling requires stress relief due to an excavation, accessibility to fresh water and an outward salt concentration gradient from the pore fluid of the rock to the ambient fluid (chloride diffusion) (Lee & Lo, 1989). This process is a significant consideration for the tunnel design, as tunnel excavation, followed by the introduction of a significant fresh water source to the Queenston Formation would initiate swelling and could cause damage to the final liner when dewatered.

Based on laboratory measurements of time-dependent deformation by Lee and Lo (1989) and others, it is found that the horizontal swelling potential of the Queenston Formation is isotropic and that the vertical swelling potential is up to 1.6 times the horizontal swelling potential. The swelling deformation response is stress-dependent, and can be represented by a linear relationship between swelling potential and applied stress in a semi-log plot and that swelling can be completely suppressed under 4 to 5 MPa stress in the Queenston (Rigbey &
Hughes, 2007). The stress acts to confine the sample, which prevents the expansion of the pore spaces. However, deterioration of the sample may still be possible, even under confined conditions, if a fresh water source is allowed to diffuse the salt ions.

![Graph showing swell strain in Y-axis (%) over days for samples S1, S2, S3a, S3b from Queenston Siltstone, January 30 - May 2, 2007.](image)

**Figure 3.14:** Free swell test results for samples of the Queenston Formation. The samples were taken from a drill hole directly below the Whirlpool – Queenston disconformity. Inset graph is for the first 10 days of swelling (original data from testing conducted by the author).

**Table 3.11:** Swell potentials (for samples tested by the author between January 30 to May 2, 2007.

<table>
<thead>
<tr>
<th>Sample</th>
<th>S1</th>
<th>S2</th>
<th>S3a</th>
<th>S3b</th>
</tr>
</thead>
<tbody>
<tr>
<td>Swell Potential (% strain / log cycle)</td>
<td>0.022</td>
<td>0.270</td>
<td>0.630</td>
<td>0.011</td>
</tr>
</tbody>
</table>
The Queenston formation was recently tested using a free swell cell. As part of this
research an investigation into the swelling process was studied and the results of this work can be
found in Appendix A. The results, Figure 3.14, indicate that rapid expansion occurs within the
first three days and following that, the rate of swelling gradually decreases over time. Table 3.11
indicates that the swell potential for the samples tested ranges from 0.011 to 0.63 % strain / log
cycle. The total strain at the end of swell testing, shows little variation which is to be expected as
the samples come from the same approximate depth and location. The local variation in swell
potential between 10 and 100 days can be explained due to crack propagation which begins
around day 10 and reaches its maximum after 100 days according to Lee and Lo (1989), but is
sample dependent, which causes variation. The free swell test results are plotted in Figure 3.14.

Two of the samples were left in the cell longer and these samples began to show signs of
increased swelling strain rate around 80 days. This is consistent with reports by Hawlader et al.
(2003) who indicated that this is likely to occur due to swelling induced crack propagation
through the samples, which increases the surface area for swelling to occur. Unfortunately the
testing had to be stopped due to laboratory constraints.

3.2.2 Failure Modes in Sedimentary Rocks

Failure modes in underground excavations are of two types; gravity driven and stress induced.
Both modes can occur in the same rock mass where structure and stresses are of the appropriate
orientation and concentration. Gravity driven failures utilize existing structure, which results in
block fall or sliding and/or relaxation and deflection. Typically under high stress conditions
structural features become clamped and therefore with increasing depth, structurally controlled
failures can be expected to occur less frequently (Hoek & Brown, 1982). Stress induced failure can result in slabbing or spalling and will typically occur when the stresses around the excavation are less than roughly one fifth the unconfined strength (UCS) (Hoek & Brown, 1982).

3.2.2.1 Gravity Driven, Structurally Controlled Failure

For structurally controlled failure to occur three or more planes of weakness must intersect. The resulting block can either fall directly or slide on one or more of the weakness planes. Hoek and Brown (1982) give details of analyzing these failure modes and developing support.

In horizontally laminated sedimentary rocks, one of the planes of weakness are bedding planes. The bedding planes, whether intact or an open joint, can be continuous for many meters and wedges can easily form with the intersection of other vertical structures, such as joints. Vertical and inclined joint surfaces can be common in sedimentary rocks due to tectonic deformation, stress relief or near surface glacial rebound. These joints will often be discontinuous and stepped due to the pre-existing bedding planes. This can result in smaller wedge volumes for direct fall out than when joints are continuous. However, if vertical structure closely parallels the excavation, then large wedge volumes can also occur. In horizontally laminated ground, the orientation of secondary structures such as vertical joints is of primary concern, as the primary plane of weakness will follow the excavation at all tunnel orientations.

Sliding blocks can also occur in horizontally laminated ground; however these are less common in the crown and are usually associated with the sidewall or haunch due to bedding being horizontal. The horizontal laminations make blocks preferentially slab-like in shape, when the joint frequency and number are low. The sliding wedges in the sidewall will be a result of
sub-vertical structure, such as joints, which dip into the excavation. Sliding wedges can be more easily supported in TBM excavations because they typically have longer stand up time than falling wedges. The advantage of the TBM is that these wedges can be progressively supported, if noticed, as they are exposed. In drill and blast operations, wedges can be fully exposed and collapse in one round of blasting leaving a large void to fill with concrete.

The voussoir model has been used to describe gravity driven delamination in parallel foliated, bedded or jointed rock masses (Diederichs & Kaiser, 1999). The parallel features, laminations, can be the result of sedimentary layering, extensile parallel jointing, metamorphic or igneous flow processes or excavation parallel fractures. The voussoir model assumes that vertical structure also cuts through the laminations or that support can be installed, such that a compression arch can be formed to transmit the beam loads to the abutments (Diederichs & Kaiser, 1999). Failure of a voussoir beam can be caused by buckling or snap-through failure, compressive crushing failure at the mid-span and/or abutments, abutment slip and diagonal fracturing, as illustrated in Figure 3.15. The iterative calculation method, reported by Diederichs and Kaiser (1999), does not account for in-situ stresses acting on the voussoir beam.
Figure 3.15: Voussoir beam failure modes. a) snap-through b) crushing c) sliding and d) diagonal cracking (after Diederichs & Kaiser, 1999).
3.2.2.2 Stress Induced, Gravity Assisted Failure

Tensile failure and shear failure can occur in all rock types around underground excavations. In sedimentary rocks, laminations act as nucleation points for fracture propagation and tensile fractures will generally follow these bedding planes. Shear failure from two beams sliding past one another can also occur and can be enhanced when weak inter bedding (shale partings) are present. Shear failure can also occur in the intact rock between the laminations, resulting in fracture growth perpendicular to the laminations. The laminations generally limit the perpendicular fracture growth to several laminations before a step occurs to offset the fracture before continued growth.

In horizontally laminated ground, with high horizontal stress, compressive stresses will be concentrated in the crown and invert and tensile stresses will be concentrated in the side walls as the stresses flow around the excavation. The compressive stresses in the crown and invert can result in shear failure of the rock mass, which can create slabs and spalls. The broken rock fragments will generally be rectangular in shape, conforming to the laminations. At the sidewalls the horizontal nature of the laminations can minimize the tensile damage which would otherwise occur in a homogeneous isotropic rock masses.

Generally speaking stress induced failure in sedimentary rocks, particularly if horizontally laminated, will result in an over broken profile which will have a stepped appearance due to the laminated nature of the rock mass. If the stress levels are high then stress notches can form with smooth sides, similar to those observed in highly stressed granitic rocks at the Underground Research Laboratory (Hajiabdolmajid et al., 2002), otherwise stress induced failure will be limited to a certain height and be truncated by a lamination to produce a flat back to the over broken zone, parallel to the laminations.
These failure modes should be anticipated in sedimentary rocks and will depend on the type of rock, the structure present and the stress levels. At the Niagara Tunnel Project, the rock formations range from limestone, to sandstone, to shale and siltstone. The joints are generally widely spaced and vertical and the stress ratio \( \sigma_h / \sigma_v \) is in the order of 3-5. The observations from the Niagara Tunnel Project and the associated failure modes will be discussed in the next section.

### 3.2.3 Failure Modes Experienced at the Niagara Tunnel Project

The Niagara Tunnel Project is being excavated in the Silurian strata after passing under St. Davids Buried Gorge in fall 2008 and ascending to the Queenston – Whirlpool contact in spring of 2009. Overbreak was encountered in the Rochester, Neagha, Grimsby, Power Glen and Queenston Formations, with minor loosening and fracturing in other units along bedding planes. The various rock mass properties were presented earlier in this thesis (section 3.2) and by Perras and Diederichs (2007) and Rigbey and Hughes (2007). Mazurek (2004) gives a good overview of the rocks of Southern Ontario in a report for nuclear waste storage. The formations where overbreak occurred had RMR values of less than 65 and UCS values of less than 50 MPa.

The stress ratio, \( K_\text{o} \), in the Queenston ranges from 3 to 6, with the principal stress ranging from 9 to 23 MPa (Yuen et al., 1992). Major physiographic features in the area, such as the Niagara Gorge, St. Davids Buried Gorge and the Niagara Escarpment, locally influence the stress field. The portal, at the tunnel outlet, to the Niagara Tunnel is located within a block of rock bounded by the physiographic features mentioned above, which has reduced the stress from the regional levels. Most of the jointing in the project area is also associated with the
physiographic features, such as observed vertical jointing and shear features under St. Davids Buried Gorge. With the high horizontal stresses, the instability is focused in the crown, haunch and invert. Sidewall overbreak was limited in the upper units above the Queenston Formation and associated with random vertical jointing. Overbreak at the Niagara Tunnel Project can be broken down into 4 zones, as illustrated in the tunnel longitudinal section of Figure 3.16. The interpretation of the failure modes presented below is based on the observations of the author over the first 2 km of the tunnel and is the author’s opinion.

Figure 3.16: Long section of the Niagara Tunnel Project showing major geological groups and the original tunnel alignment. Overbreak zones are 1) Formations above the Queenston, 2) Whirlpool – Queenston contact, 3) St. Davids Buried Gorge, and 4) Regional stress field (from Perras and Diederichs, 2009, original data courtesy of Ontario Power Generation).
3.2.3.1 Overbreak in the Formations Above the Queenston

Overbreak in the formations above the Queenston were limited to induced fracture growth at shale partings, thin bedding plane fall out in the haunch, gravity slabbing, wedge failure and stress induced slabbing. These failure mechanisms will be discussed in the formations where they were observed.

The Rochester Formation is a dark grey calcareous shale – dolomite interbed, which is roughly 19 m thick at the project site. Bedding is difficult to observe in hand sample, with the exception of occasional slight colour changes. The formation is a stiff, yet lower strength rock mass, which has very few joints. Gypsum nodules are present in the rock mass and can act as nuclei for induced fracture growth.

The overbreak in the Rochester Formation was limited to the crown and invert, with no sidewall damage observed. In the haunch area, high angle induced fractures cut across the bedding, creating slabs of rock and a stepped edge to the overbreak profile, as shown in Figure 3.17. Evidence of shear failure in the haunch area was observed as rock split across bedding to form slabs parallel to the excavation profile. A flat back was created by horizontal tensile fracture growth parallel to the bedding.
Figure 3.17: Top picture shows overbreak with stepped edge, bottom left shows haunch rock fractures across bedding induced by stresses and bottom right shows water inflow from overlying Lockport Formation (photos courtesy of Ontario Power Generation).

The competent Lockport and Irondequoit limestones, above and below the Rochester, respectively, and the low ground cover minimized the extent of the overbreak in this unit. However water inflows (Figure 3.17) from the overlying Lockport, caused adverse working conditions for the tunneling crew, well into the Rochester since the overbreak extended back to the contact.

The Neagha is a fissile (Figure 3.18) green shale, roughly 1.8 m in thickness and is the weakest unit crossed by the tunnel. The bedding thickness is less than 1mm and the fissility leads to near zero tensile strength. The Neagha broke back to the overlying Reynales limestone, when exposed at the back of the roof shield of the TBM (Figure 3.18), due to gravity. The unit is extremely weak prior to excavation and the observed overbreak was related to flexural bending.
Figure 3.18: Left photo shows fissile nature of Neagha (between 0 – 10 cm) below the Reynales limestone. Right photo shows typical Neagha overbreak from the Niagara Tunnel Project (photos courtesy of Ontario Power Generation).

Figure 3.19: Shale interbed loosening in the Thorold Formation (photo courtesy of Ontario Power Generation).
and gravity fall out, similar to the voussoir analogue (Diederichs & Kaiser, 1999). The competent units above and below the Neagha controlled the depth and width of the overbreak area.

The Thorold Formation is a light grey sandstone with interbedded dark grey shale. It is a strong unit roughly 2.7 m thick at the project site. Generally speaking there was negligible overbreak in the Thorold and only minor loosening where shale beds day lighted in the haunch area, as seen in Figure 3.19. This fall out was gravity driven, as the shale layer was unable to support its own weight and the disk cutters removed the loosened shale layer.

The Grimsby Formation is irregularly bedded sandstone with dark red shale interbeds (Haimson 1983). Cross bedding is a common feature within the Grimsby and being predominately competent sandstone, the overbreak was minimal. Minor loosening and bedding parallel fractures were observed to open 1-2 mm in the haunch area and fall out only occurred along thick shale layers (0.1 to 0.2 m) as they approached the tunnel crown, as seen in Figure 3.20. The weak shale layer promoted bed parallel fractures to develop, but the competent sandstone layers minimized the depth of overbreak. Some minor stress induced failure occurred where shale bed movement caused the thin shale layer to pop out from the excavation surface, as shown in Figure 3.20.

The Power Glen Formation can be divided into two units. The upper unit is a light grey sandstone with grey interbeds of shale and the lower unit is predominately grey shale with interbeds of light grey sandstone. The upper unit had minor instability and dilation of beds in the crown, with minor overbreak along shale beds, similar to the Grimsby Formation. The lower unit had significant overbreak in the haunch area, but was of limited height vertically above the crown due to the more competent upper unit (Figure 3.21).
Figure 3.20: Left photo shows dilation and minor fall out of 0.2 m shale beds in the crown. Right photo shows stress induced pop out failure of 200 mm thick shale bed in the Grimsby Formation. The dark bands are shale layers and the light bands are sandstone. The field notebook is resting on top of the shale layer which has popped out 0.02 – 0.03 m (photos courtesy of Ontario Power Generation).

The width of the overbreak was also controlled by the stiff Whirlpool sandstone below, which reduced flexural bending of the beds in the lower Power Glen unit and minimized the tensile failure and overall overbreak dimensions.

Two separate gravity wedges were encountered and supported in the tunnel where random, long continuous vertical joints ran 20-30 ° off the tunnel axis. These gravity wedges were held in place in part because a third joint surface for to create fall or slide out conditions was missing, and the compressive strength of the sandstone layers was high enough to prevent direct snap through. Some buckling of these sandstone layers did occur prior to support installation, but once supported the wedge was stable. The weak shale beds in the upper Power Glen and the vertical joint created the geometry of the wedge, is illustrated in Figure 3.21.
Figure 3.21: Upper photos show localized haunch failure in the Power Glen. Bottom shows supported wedge in the Power Glen Formation with inset illustration of failure surfaces (photos courtesy of Ontario Power Generation).
The overbreak in the units above the Queenston Formation, (zone 1, Figure 3.16) was influenced by the interbedded nature of the rock mass and the units above and below. Aside from added influences as mentioned above, the overbreak in zone 1 (Figure 3.16) was controlled by gravity, the high horizontal in-situ stresses, and local structure.

3.2.3.2 Overbreak Observed in the Queenston Formation

Overbreak in the Queenston Formation can be divided into three zones, as illustrated in Figure 3.16 (zones 2-4). Within each of these zones the overbreak can be generalized into one failure mechanism; however there is a transition area between each zone which is difficult to determine precisely.

The contact between the Whirlpool and Queenston Formations is a disconformity, an erosional surface parallel to the formation bedding, marking the transition from Ordovician (Queenston) to Silurian (Whirlpool) time. The erosional surface represents a gap in deposition when weathering, uplift and other degradation processes were occurring. The stiffness contrast between the Whirlpool and the Queenston creates a local stress shadow below the contact, reducing the stress levels slightly. The stress, in combination with local jointing and the presence of the strong Whirlpool above, influenced the overbreak size and shape in zone 2 (Figure 3.16). The overbreak was observed to break back to the overlying Whirlpool Formation (Figure 3.22) to a maximum depth of 1.4 m, at which time forward spiling was used to control overbreak and advance the tunnel.
Overbreak approaching St. Davids Buried Gorge was limited to less than 1 m due to the stress field modified by the physiographic features, and once reaching the structural influence zone (zone 3, Figure 3.16) of St. Davids Buried Gorge, overbreak reached depths in the order of 3 m. Vertical jointing, spaced 2-3 m, and horizontal and inclined shear surfaces were observed under St. Davids Buried Gorge. The vertical jointing remained clamped due to the stress concentrations in the crown, as shown in Figure 3.23 and had minor influence on the overbreak depth. The horizontal and inclined shear surfaces also had some effect on the overbreak, although it was difficult to determine when in fact a sheared surface existed above the crown.
As the tunnel advanced away from the influence of St. Davids Buried Gorge, the regional high horizontal stresses were encountered and overbreak continued to be in the order of 2-3.5 m in depth at the crown, with maximums approaching 4 m. The overbreak zone was characterized by steep sides where induced tensile fracturing was observed in the upper haunch area, with horizontal induced fracturing above the crown elevation, creating a plane dipping towards the face, likely due to stress rotation near the excavation face.

Figure 3.23: Clamped vertical joints in the crown in the Queenston Formation, under St. Davids Buried Gorge (photo courtesy of Ontario Power Generation).
Figure 3.24: Notch shaped overbreak in the Queenston Formation from the Niagara Tunnel Project. Top photo shows 3.78 m of overbreak. Bottom left shows view looking back over the TBM, bottom right shows haunch area and slabbing rotating from near vertical to horizontal in the crown. (photos courtesy of Ontario Power Generation).
A consistent notch shape (Figure 3.24), skewed to the left was observed, likely indicating a high stress ratio with the major principal stress orientation slightly inclined from horizontal. Failure in the crown and invert, due to the stress concentration, resulted in first the haunch rock area yielding, followed by the crown and invert degradation. The failure in the crown and invert were the dominant locations of yielding, however minor sidewall damage also occurred.

Sidewall tensile fracturing also occurred and was observed only on the left hand side wall. This could be caused by the horizontal intermediate principal stress, \( \sigma_2 \), being sub-parallel to the tunneling direction, or rotation of the cutterhead. The localization of the tensile fractures would result from the stress flow around the tunnel face. As the intermediate principal stress flows around the excavation face, a low confinement zone is created on the right hand side and high confinement zone on the left hand side. The high confinement promotes tensile fracturing parallel to the minor principal stress, \( \sigma_3 \), creating excavation parallel fractures. These fractures generally did not coalesce to form spalls and were localized. They resulted in distinct surfaces 2-3 cm deep, spaced 10-20 cm apart, as shown in Figure 3.25, and produced negligible overbreak, as the fractures were only in the order of 1-2 m in length. These fractures could also be caused by the rotation of the cutterhead. The cutterhead spins in a clock-wise manner, creating a left-hand and downward pull. This could cause increased damage on the left hand side of the tunnel, as the cutterhead oscillates back and forth during mining cycles. Although the surface fracturing was not observed continuously throughout the Queenston, suggesting that the intermediate principal stress was playing the dominate role in the damage. As the tunnel turned into the high horizontal stress, the fracturing on the left hand side ceased.

Local and minor in volume, overbreak occurred where vertical joints intersect the tunnel, as shown in Figure 3.25. The failure process here is induced by movement on the joint and the
loosening during the excavation process removes the material where the wall rock is thinnest. When the joint intersects the tunnel sidewall in the haunch area, gravity assists in loosening the material for removal due to the excavation process or subsequent fall out at a later time.

The variable ground conditions at the Niagara Tunnel Project raise the question, when do laminations within the rock mass become an important consideration for design purposes? How does the variation in lamination thickness affect the stability of the excavation? The units above the Queenston Formation range in thickness from 2 to 20 m and the stability of the excavation appears to be related not only to the units at the excavation face, but also to those above and below the tunnel. The Queenston Formation is over 300 m thick at the project site and the Niagara Tunnel is excavated into roughly the top 60 m. This provides an opportunity to study an anisotropic rock mass outside the influence of other formations. The numerical modeling presented in the following chapter will address these questions for a single intact rock type, which represents a clay shale.
Figure 3.25: Top left and right photos show stress induced fractures on the left hand sidewall of the Niagara Tunnel Project. The bottom left photo localized overbreak associated with a vertical joint in the Queenston Formation (photos courtesy of Ontario Power Generation).
Numerical modeling techniques are used to understand and/or predict the behaviour of excavations for application in design and research. They are often used by geotechnical engineers to understand the stability and ground response of the required excavation to determine the limitations of the design parameters. There are many types of numerical methods available, such as Finite Element Methods (FEM), Finite Difference Methods (FDM), Boundary Element Methods (BEM) and Distinct Element Methods (DEM), and each has its strengths in potentially aiding the geotechnical engineer for a particular problem. In the modeling presented here, DEM and FEM methods have been used to determine the relationship between lamination thickness and the stability of large diameter circular excavations.

4.1 The Distinct Element Method
The distinct element method allows for the representation of discontinuities and these create discrete elements by dividing up the domain. This is often referred to as a discontinuum numerical method. The discrete elements represent blocks of rock, which can either be rigid or deformable. Both the discontinuities and the blocks, when deformable, can be assigned failure criteria. The discrete elements are allowed to displace and rotate along the discontinuities in response to applied forces of motion. Complete detachment and new contact recognition allows

Included in part in the conference paper by Perras & Diederichs (2009c).
for realistic block movement beyond continuum methods. The governing laws of motion are based on Newtonian mechanics and when one block contacts its neighbor, it creates a chain reaction of movements throughout the model. The solution is based on contact and impact between multiple bodies (Pande et al. 1990) and time steps, which are sufficiently small to detect contacts between blocks, are used to cycle through to equilibrium. Equilibrium is determined, typically by the user, when movements around the problem area are small enough to be considered static. Each block is an individual entity and is not rigidly connected to its neighboring blocks, which is the primary difference between DEM and the FEM. The blocks communicate using boundary contacts which are allowed to change with each cycle (Pande et al. 1990). For an explanation of governing equation and details of the DEM, the reader is referred to Pande et al. (1990) and Itasca (2000). The DEM method is used in engineering to determine the failure mode of fractured and jointed rock masses by including both the rock blocks and the discontinuities.

UDEC, a DEM program offered by Itasca, was used in this study to represent the laminations as discrete elements. The laminations were represented by discontinuities which create beams of rock, in between the laminations, which are deformable blocks. This also allowed for flexural bending of the laminations and plastic yielding of both the discontinuities and the rock beams. The primary use of the UDEC software was to validate the failure mechanism modelled by the finite element code Phase2, by RocScience, and to ensure that the failure process and results were repeatable with alternative software. UDEC is the traditional software in North America for modeling jointed rock masses, however the latest version of Phase2 (version 7) offers a very robust joint network tool which allows for automated joint
generation. UDEC remains a valuable tool for research since the FISH (Itasca, 2000) programming language allows full access to the software code for manipulation.

4.2 The Finite Element Method

The finite element method uses continuum mechanics to solve complex problems, where an analytical solution is cumbersome and time consuming or unavailable. The problem geometry is broken down into small zones, called elements, which have nodes at the intersection of zone edges. The nodes connect each element together to form a continuum and the nodes are the central location where data is stored and updated as the stresses or forces are applied to the problem. The assembly of elements is called a mesh and different sections of the mesh can contain different material properties. As the mesh density increases, a better approximation of the mechanical equations across the problem is achieved. However, as the mesh density increases, the calculation time increases and a balance between the final results and the processing time must be optimized.

The finite element method can be used for any physical problem which is governed by differential equations (Pande et al., 1990). The displacements of each element are used to determine the strain of each element and from the strains, using material properties and stress-strain relationships, the stresses can be determined throughout the problem geometry. The formulation is controlled by incremental strains, each strain step being small enough to cause changes to the neighboring element and minimize shock to the neighboring rock mass. Calculations are continued until a balance between the driving and reaction forces are achieved, which is called unbalanced forces in Phase2 (RocScience, 2008), and represents the unbalanced energies within the system. This is only used when plastic materials are being model led, as the
solution to the set of differential equations is only exact for elastic materials. The tolerance of the unbalanced force can be adjusted by the user depending on the complexity of the problem. However, for comparison between different models, the unbalanced force tolerance level should be the same.

Phase2, by RocScience, is a finite element program which is used for geomechanical analysis. The latest version of Phase2, version 7, offers a joint network tool as mentioned above. This tool enables the user to generate a network of joints with ease. For this reason the program has been used to represent a horizontally laminated rock mass, with the joint elements representing intact laminations for the purposes of this study. The joint network is essential to capture the mechanical motion of a horizontally laminated rock mass as it deforms in response to an excavation.

4.3 Numerical Accommodation of Anisotropic Rock Masses

The majority of sedimentary and metamorphic rocks are anisotropic due to parallel features which are inherent within these rock types. These parallel features give rise to different rock mass properties parallel and perpendicular to the feature and create differential stress and strain responses depending on the direction of loading. In sedimentary rocks, anisotropy is a function of bedding planes, laminations, layering and/or parallel jointing (Saroglou & Tsiambaos, 2007).

Since anisotropy is created by varying strength properties in orthogonal directions, properties which affect the stiffness of the rock mass will have the largest effect on the degree of anisotropy. Chappel (1990) found that anisotropy is both modulus and stress dependent, and therefore not all rock masses with parallel structure or fabric will exhibit the same degree of
anisotropic behaviour, and similar rock masses under different stress regimes will not necessarily behave in a similar manner.

Research into anisotropic strength in the early 1960’s was focused on determining the Unconfined Compressive Strength (UCS) or triaxial strength of intact samples for various angles between the plane of anisotropy and the loading direction (Donath, 1961 and Hoek, 1964). Common curves, such as that illustrated in Figure 4.1, are found in most rock mechanics texts, such as Brady and Brown (2006) for example, showing a relationship between the axial strength and the angle of the normal to the plane of weakness. The curve in Figure 4.1 shows a zone of slip on the weakness plane, associated with the friction angle, and the peak failure stress is associated with failure through the intact portion of the sample.
Figure 4.1 Graphical representation of a triaxial compression test showing the orientation of the normal to the plane of weakness with respect to the primary loading direction. b) variation of peak strength at constant confining pressure with the angle of inclination of the normal to the plane of weakness. Images modified from Brady and Brown (2006)

4.3.1 Elastic Methods

Elastic solutions are possible, using transversely isotropic or orthotropic material properties (Brady & Brown, 2006) which treat the rock mass as having strength and stiffness planes, within which these values remain constant. This is illustrated in Figure 4.2 below, where the cube is divided by parallel lines indicating the planes of isotropic behaviour.

For a transversely isotropic material there are five independent elastic constants and orthotropic materials have nine (Brady & Brown, 2006). In practice it is expensive to determine
all the elastic constants necessary to completely characterize a transversely isotropic or orthotropic material for a project and as such there has been little advancement in plastic yield constitutive models for transversely isotropic or orthotropic materials.

The properties of a transversely isotropic material can be found using the laboratory properties for intact rock samples parallel to the plane of isotropy and determining the normal ($K_n$) and shear ($K_s$) stiffnesses for the laminations or joints. The equations for determining the necessary input parameters are given in Table 4.1 (Brady & Brown, 2006) and have been arranged assuming the plane of isotropy is horizontal (i.e. the $1_z$ plane) as illustrated in Figure 4.2.

Figure 4.2: Illustration of plane of transverse isotropy with axes labeled according to the convention in Phase2 (RocScience 2008).
Transversely isotropic elastic solutions are available in finite element modeling software and Phase2, by RocScience, was used in the work presented below. Using the equations of transverse isotropy given in table 4.1 the rock mass properties were determined for a series of models and compared to isotropic elastic results. Vertical crown deflections were used as an indicator for model performance and comparing the results for a series of lamination thicknesses, the resulting deflections at the crown are greater for the transversely isotropic models below lamination thicknesses of 1100 mm for a 16 m diameter tunnel, at 150 m depth and at Ko ratios of 1, 2, and 3 as shown in Figure 4.3.
Table 4.1: Table of elastic properties necessary for input into numerical modeling software for a transversely isotropic elastic material (Brady & Brown, 2006). Note that the subscripts denote the reference axes as shown in Figure 4.2.

<table>
<thead>
<tr>
<th>Elastic Property</th>
<th>Method of Determination</th>
<th>Equation Reference Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>Poisson’s ration, $\nu_{1z}$</td>
<td>$\nu_{intact}$ (laboratory testing)</td>
<td>-</td>
</tr>
<tr>
<td>Youngs Modulus, $E_{1(z)}$</td>
<td>$E_{intact}$ (laboratory testing)</td>
<td>-</td>
</tr>
<tr>
<td>Angle</td>
<td>angle between horizontal and plane of isotropy</td>
<td>-</td>
</tr>
<tr>
<td>Shear Modulus, $G_{1(z)}$</td>
<td>$\frac{E_{1(z)}}{2(1 + \nu_{1z})}$</td>
<td>Equation 4.1</td>
</tr>
<tr>
<td>Youngs Modulus, $E_2$</td>
<td>$\frac{1}{E_{1(z)}} + \frac{1}{K_n T}$</td>
<td>Equation 4.2</td>
</tr>
<tr>
<td>Poisson’s ration, $\nu_{12}$</td>
<td>$\frac{E_2}{E_{1(z)}} \nu_{1z}$</td>
<td>Equation 4.3</td>
</tr>
<tr>
<td>Shear Modulus, $G_{12}$</td>
<td>$\frac{1}{G_{1(z)}} + \frac{1}{K_p T}$</td>
<td>Equation 4.4</td>
</tr>
</tbody>
</table>
Figure 4.3: Comparison of vertical crown deflections for elastic models of 16 m diameter circular excavations at 150 m depth for rock properties associated with $E_i = 4$ GPa. Results are from Phase2 modelling.
In order to develop an anisotropic (or transversely isotropic) plastic modeling method the anisotropic elastic solution should match closely to the proven transversely isotropic elastic method. Essentially the anisotropic elastic method incorporates discrete laminations to account for the transverse isotropy. To compare with the transversely isotropic elastic method, the laminations and the intact rock, between the laminations, were modeled elastically. The laminations account for transverse modulus out of the plane of isotropy. The specific model setup and procedure will be discussed in more detail in Section 4.3.4.

The stress contours around a 16 m diameter tunnel for all three elastic methods, isotropic elastic, transversely isotropic elastic and anisotropic elastic, are compared in Figure 4.4. The stress contours for the transversely isotropic and anisotropic elastic methods compare very closely and show the same vertical truncation and lateral expansion compared to the isotropic case which is more bulbous in shape. Further confirmation of the validity of the anisotropic elastic method is given in Figure 4.3, where the vertical crown deflections are compared. The transversely isotropic and the anisotropic methods show very similar crown deflections at all lamination thicknesses and at all Ko ratios. The crown deflections are larger at lower Ko ratios than those at higher values, because the confining stress at the crown is less when the Ko ratio is low. The lower confinement means that the rock mass in the crown is more susceptible to gravity induced relaxation and deflection because it is slightly less stiff than the same rock mass at higher confinement. Further validation of the anisotropic method is necessary and will be discussed in greater detail in the next chapter.
4.3.2 Traditional Plasticity Methods

The state of engineering practice is to model a rock mass as an isotropic material. The rock mass is evaluated using a classification scheme, as discussed in Chapter 3, and strength properties are determined by laboratory testing. The laboratory test results are reduced to incorporate the heterogeneity of the rock mass using empirical relationships. One such relationship for modulus is presented by Hoek and Diederichs (2006) to determine a deformation modulus, $E_{mr}$, for the rock mass using GSI, USC, $E_i$ and $m_i$. This method has limitations when applied to anisotropic conditions.

Traditional plasticity methods include, elastic perfectly plastic, elastic brittle and strain softening. Another method, elastic-brittle, strain dependent – cohesion weakening and friction softening, has been applied to brittle materials with good success (Diederichs, 2007) however,
this method has high mesh and boundary condition sensitivity at high in-situ stress ratios and is not included in this discussion.

Elastic perfectly plastic means that when the stress-strain strength envelope is exceeded, the rock mass behaves plastically without a drop in peak strength. In the plastic region, the rock mass is assigned the peak strength properties once it exceeds the strength envelope. Until this point, the rock behaves elastically following the prescribed stress-strain slope defined by the modulus. When the strength envelope is exceeded a flow rule is necessary to bring the stress levels back to the strength envelope. The flow rule can be associated and when the strength envelope is exceeded, this association is used to bring the stresses back below the envelope or the flow rule can be non-associated and the stresses return via a linear function. For example, Phase2 uses a non-associated flow rule when the dilation parameter is set to zero or an associated flow rule when dilation is included. In UDEC, tensile failure is non-associated and compressive failure is associated, as illustrated in Figure 4.5. The dilation angle is used to bring the stresses back below the failure envelope, when exceeded.
Figure 4.5: Flow rules used in UDEC (Itasca 2000) when the tensile (non-associated domain 2) and compressive (associated domain 1) strength envelope is exceeded.

The elastic perfectly plastic method is the simplest plasticity model, since there is no need to determine residual rock mass strength values. This method was adopted for this research to minimize the number of variables. A comparison between the traditional isotropic plasticity and anisotropic plasticity methods are discussed in Section 4.3.4.

Determining residual rock mass strength values can be difficult and no recommendations from the scientific community have been adopted for consistent use, although Cai et al. (2006) have proposed a system based on a residual GSI value. In the elastic brittle method, when the
elastic strength of the rock mass is exceeded, the rock mass strength is dropped instantaneously to residual strength parameters to simulate rock mass damage and create a weaker rock mass in the yield zone. The strain softening method allows for a gradual reduction in the strength parameters as the strains increase.

For simplicity, the elastic perfectly plastic method was adopted for both the isotropic plastic models and the anisotropic plastic models. Further research is necessary to determine the effect of residual strength parameters and dilation for an anisotropic material and is not dealt within this research.

4.3.3 Ubiquitous Joint Method

The ubiquitous joint method employs failure criteria for an oriented weak plane and assumes that this weak plane can exist everywhere within the model domain. The UDEC code offers a ubiquitous joint model and allows for plastic yield both in the rock and on oriented weak planes. The Phase2 code offers a post-analysis ubiquitous joint calculation method (RocScience, 2008). It can be used to refine strength factor contours, by incorporating ubiquitous joint parameters; however these parameters have no effect on the stresses or displacements.

By definition, a ubiquitous weak plane exists within every zone of the model, as they are not assigned a particular spacing or location. In UDEC, during stepping, the model is first checked for failure within the rock, using the constitutive model prescribed by the user. This is followed by checking for failure of a Mohr-Coulomb plane of weakness. Since a specific location is not assigned, failure can occur throughout the model both within the rock and on the oriented weak plane. This can over-estimate the model yield zone for equivalent lamination
thickness as compared to the results from other methods (Figure 4.6). The current computational limit for lamination thickness, in Phase2, is around 200 mm.

If the lamination thickness were in the order of 1-10 mm, then the ubiquitous joint method, such as offered in UDEC, may more adequately represent the anisotropic material. Computational memory restrictions inhibit comparison of the ubiquitous joint method with the new anisotropic plasticity method, to be discussed below, since the smallest lamination thickness possible, in Phase2, is 160 mm for large diameter excavations. Small scale models, such as laboratory tests, potentially could be used to compare the ubiquitous joint method to the new anisotropic plasticity method; however this is beyond the scope of this research.
Figure 4.6: Ubiquitous joint model results from UDEC for rock mass properties associated with $E_i = 4$ GPa and for tunnel diameter of 16 m at 150 m depth with a Ko ratio of 3. Plastic yield extends up to 65 m either side of the sidewalls.
4.3.4 Anisotropic Perfectly Plastic Method

A new method of discrete anisotropy has been tested in this research to compare with traditional plasticity methods and to determine when horizontal laminations become an important consideration for excavation stability at shallow depths. To induce anisotropic behaviour, horizontal laminations were modeled using joint elements.

By modelling horizontal laminations with joint elements, the rock mass behaviour is now controlled by both the intact rock properties between the joints, and the joint properties themselves. The joint elements reduce the rock mass modulus in the vertical direction and allow for greater joint parallel displacements and for beam deflections, similar to that of a laminated voussoir beam. This behaviour is important to capture the true mechanical behaviour of an anisotropic rock mass.

For the purposes of this research, the geometry of the excavation has been fixed, so that a clear understanding of the influence of increasing lamination thickness on the excavation behavior can be determined. Five different intact rock properties were tested with corresponding lamination properties, as outlined in Table 4.2. The rock beams use a Hoek-Brown failure envelope, with perfectly plastic strength criteria and the laminations use a Mohr-Coulomb failure envelope. The model runs were also tested at horizontal to vertical stress ratios (Ko) of 1, 2 and 3 at 150 m depth, as well as Ko=1 for depths of 25, 75, 300 and 450 m. Dilation has not been included in the modeling and all materials are perfectly plastic.

A joint network, a new feature in Phase2 version 7 by RocScience (2008), was used around the excavation to model the laminations. To ensure compatibility between the laminated area and the non-laminated area, as well as to relate the isotropic models to the anisotropic models (laminated), a relationship for transversely isotropic elasticity was used to scale the
modulus, accounting for the stiffness ($K_N$) and spacing of the laminations ($T$), see Figure 4.7 for an illustration of the model setup. The laminations are joint elements, which divide the rock into rock beams, as shown in Figure 4.7.

### Table 4.2: Rock and lamination properties used in this research.

<table>
<thead>
<tr>
<th>Intact Modulus (GPa)</th>
<th>Beam GSI</th>
<th>Beam Modulus (GPa)</th>
<th>UCS (MPa)</th>
<th>$m_i$</th>
<th>Cohesion (MPa)</th>
<th>Tension (MPa)</th>
<th>Friction (degree)</th>
<th>Normal Stiffness (MPa/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>70</td>
<td>2.93</td>
<td>40</td>
<td>7</td>
<td>0.13</td>
<td>0.30</td>
<td>25</td>
<td>25000</td>
</tr>
<tr>
<td>6</td>
<td>70</td>
<td>4.40</td>
<td>60</td>
<td>9</td>
<td>0.21</td>
<td>0.35</td>
<td>27</td>
<td>50000</td>
</tr>
<tr>
<td>8</td>
<td>70</td>
<td>5.86</td>
<td>80</td>
<td>11</td>
<td>0.29</td>
<td>0.38</td>
<td>28</td>
<td>75000</td>
</tr>
<tr>
<td>10</td>
<td>70</td>
<td>7.33</td>
<td>100</td>
<td>13</td>
<td>0.37</td>
<td>0.40</td>
<td>29</td>
<td>100000</td>
</tr>
<tr>
<td>20</td>
<td>70</td>
<td>14.66</td>
<td>150</td>
<td>17</td>
<td>0.6</td>
<td>0.46</td>
<td>31</td>
<td>150000</td>
</tr>
</tbody>
</table>
Using these parameters and the beam modulus, $E_{\text{beam}}$, an equivalent non-laminated rock mass modulus, $E_{\text{rm}}$, can be equated using equation (4.5) below (Brady & Brown, 2006).

\[
\frac{1}{E_{\text{rm}}} = \frac{1}{E_{\text{beam}}} + \frac{1}{K_N T}
\]

(4.5)
The rock strength properties, $m_b$ and $s$, were also scaled, such that at larger thicknesses, the equivalent properties were similar to the individual beam properties. This was done first by adjusting the GSI value, such that the modulus was the same as that calculated using Equation 4.2 and then taking the $m_b$ and $s$ values, and harmonically averaging these with the beam $m_b$ and $s$ values. This has been illustrated in Figure 4.8.

The boundary conditions can influence the model results and testing was done to ensure that the joint network radius had minimal influence on the excavation behaviour, as well as at the outer boundary. Furthermore, the mesh style and density was also tested so that the accuracy and computational time could be optimized.

The joint network radius was calibrated to optimize the computation time without sacrificing accuracy in the results. Radii of 40, 50 and 70 m were tested. It was found that the 40 m network radius interfered with the lamination yield zone, when the thickness was in the order of 440 mm. Comparing the total displacements of the 50 and 70 m radii, in Figure 4.9, the total displacements are lower for the 70 m radius. All other parameters were held constant, including the mesh density. Further testing beyond the 70 m radii at low lamination thickness ($\leq 440$) was limited by computational memory restriction. With a joint network radius of 80 m, the number of mesh elements increases significantly from 49762 to 57402 and the software crashes when trying to compute using the Gaussian elimination method, which is the fastest method available in Phase2. A conjugate gradient method is available and the 70 m joint network radius model was computed in ~8 hours with this method versus ~1.5 hours for the Gaussian elimination method of the 70 m radius. To save on computational time, the 70 m joint network radius was used with the Gaussian elimination method.
Figure 4.8: Illustration of harmonic averaging method used for Young’s modulus and Hoek-Brown parameters $m_b$ and $s$. 
To ensure that the boundary conditions did not influence the displacements at the excavation boundary, models were computed with widths of 150, 350 and 500 m. The total displacements were measured at the both the ground surface, and half way from the ground surface to the excavation, to determine the influence of the model width on rock mass hang up. Hang up occurs when the width of the model is too small and the rock mass is supported by the edges of the model. This can be partially overcome with roller boundary conditions on the sides, however a comparison, in Figure 4.10, indicates that some hang up is still occurring with a model width of 150 m. There is little difference between the 350 and 500 m wide models and as such 350 m was selected as the optimum model width.

Figure 4.9  Example of joint network radius calibration in Phase2 for a 16 m diameter tunnel with lamination thickness of 440 mm and properties associated with $E_i = 4$ GPa.
The mesh style can change the stiffness of the rock mass around the excavation and careful consideration must be taken to avoid an overly stiff mesh. When modeling a horizontally laminated rock mass, with small lamination thicknesses, the stiffness of the rock beams is influenced by the element size and the number of nodes. To eliminate the influence of element size, triangular elements with a width roughly equivalent to the lamination thickness were used to simulate low lamination thicknesses. As the lamination thickness increased, when the automatic meshing feature began to place two elements per beam, the element width was adjusted to half the lamination thickness. This process was continued such that a sharp change in the element width between successive lamination thicknesses was avoided. The stiffness of the rock beams was also controlled by the number of nodes per element. In Figure 4.11, a comparison is made.
between 6 noded and 3 noded triangular mesh elements. There is negligible influence on the total
displacement and the stress in the XX direction; however a saw toothed pattern is apparent for the
3 noded triangles for the stress in the YY direction. The saw tooth pattern is the result of a lack in
inter-beam strain capability for the 3 noded triangles. This creates inter-dependence on
neighbouring rock beams for the calculation of the stresses creating the saw toothed pattern. The
6 noded triangle allows for strains to be calculated on the mid side nodes and increases the
degrees of freedom, thereby allowing inter-beam strain to be calculated. This smoothes the YY
stress curve, since each beam can strain independently from its neighbour.

![Graph showing comparison between 6 noded and 3 noded triangular mesh elements for stresses (XX and YY) and displacements for a 16 m diameter tunnel. Model results from Phase2.](image)

Figure 4.11 A comparison of stresses (XX and YY) and displacements for 6 noded and 3
noded triangular mesh elements for a 16 m diameter tunnel. Model results from Phase2.
To determine the validity of the new anisotropic modeling method proposed above, a variety of rock and joint properties were tested, as presented in Table 4.2, to compare the methods outlined above. These were initially compared between two different software programs, UDEC by Itasca, and Phase2 by RocScience, to determine if any differences between the distinct element method (UDEC) and the finite element method (Phase2) were appreciable. Since the plastic yield zones, as compared in Figure 4.12 and Figure 4.13, are very similar between the two codes, the more user friendly code (Phase2) was used for the remainder of the research, except for validation of several models within each stress regime. The Phase 2 models for discrete anisotropy and equivalent lamination thickness isotropic models are compared between Figure 4.13 and Figure 4.14.
Figure 4.12: UDEC discrete anisotropic model results for various lamination thicknesses. Tunnels are 16 m in diameter and the rock mass properties are associated with $E_i = 4$ GPa and $K_o = 3$. 

\[ \text{280mm Thickness} \]
Plain yield stabilized

\[ \text{1200mm Thickness} \]
Multi-beam coupling

\[ \text{280mm Thickness} \]
Plastic yield stabilized

\[ \text{1200mm Thickness} \]
Multi-beam coupling

\[ \text{3600mm Thickness} \]
Stress channeling

\[ \text{8000mm Thickness} \]
Isotropic behaviour

3-Mar-09 18:31
cycle 380001
block plot
no. zones : total 222650
at yield surface (*) 836
yielded in past (X) 11476
tensile failure (o) 3089
joints now at shear limit

186
<table>
<thead>
<tr>
<th>280mm Thickness</th>
<th>1200mm Thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plastic yield stabilized</td>
<td>Multi-beam coupling</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Thickness</th>
<th>Description</th>
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</thead>
<tbody>
<tr>
<td>280mm</td>
<td>Plastic yield stabilized</td>
</tr>
<tr>
<td>1200mm</td>
<td>Multi-beam coupling</td>
</tr>
<tr>
<td>3600mm</td>
<td>Stress channeling</td>
</tr>
<tr>
<td>8000mm</td>
<td>Isotropic behaviour</td>
</tr>
</tbody>
</table>

Figure 4.13: Phase 2 discrete anisotropic model results for various lamination thicknesses. Tunnels are 16 m in diameter and the rock mass properties are associated with $E_i = 4$ GPa and $K_o = 3$. 

187
Figure 4.14: Phase 2 isotropic model results with equivalent laminated rock mass properties for various lamination thicknesses. Tunnels are 16 m in diameter and the rock mass properties are associated with $E_i = 4$ GPa and $K_0 = 3$. 
Using the vertical crown deflection for comparison, the influence of lamination thickness can be seen in Figure 4.15 for the various modeling methods used in Phase2. The model results indicate that the elastic methods under-predict the crown deflections at all lamination thicknesses, except with a Ko ratio of 1, where above a lamination thickness of 7250 mm, all model types give similar results, because there is little plastic yield occurring.

The importance of lamination slip (yielding) is illustrated in Figure 4.15 by comparing the rock plastic, joints elastic model results against the other modeling results. By preventing the laminations from slipping (remain elastic) the vertical crown deflections are suppressed to levels comparable to when no joints are used. There is significant deviation from the rock plastic, joints plastic curve when the joints remain elastic, especially at small lamination thicknesses. When comparing the rock plastic, joints plastic curve to the other modeling methods, a clear distinction in the relationship between vertical crown deflection and lamination thickness is obvious and various zones (Figure 4.15, 1-4) of behaviour with respect to lamination thickness are evident. This behaviour is not present in the other methods, as shown in Figure 4.15, and will be discussed further in the next chapter.
Figure 4.15: Comparison of modeling methods at various lamination thicknesses using vertical crown deflection. Rock Plastic, Joints Plastic = anisotropic plasticity, Rock Elastic, Joints Elastic = anisotropic elasticity, Rock Plastic, No Joints = isotropic plasticity and Rock Elastic, No Joints = isotropic elasticity. Model results are from 16 m diameter tunnels at 150 m depth and associated with rock mass properties of $E_i = 4$ GPa. Model results from Phase2.
To fully understand the behaviour and limitations of anisotropic rock masses, a series of models were computed to determine the influence of horizontal laminations on the vertical crown deflections and plastic yielding.

In Figure 4.15, from Chapter 4, the anisotropic plastic model runs (rock plastic, joints plastic) indicate that there is a unique behaviour of an anisotropic material which is not present with other modeling methods, namely isotropic plasticity (rock plastic, no joints), as indicated by the difference in the curve shapes which is to be discussed below. This behaviour is the result of the ability of the rock mass to slip laterally and deflect due to the presence of the laminations, which are allowed to yield. At specific lamination thicknesses, as defined in Figure 4.15, four different behavioural styles exist for the rock mass tested (Ei = 4 GPa) and they are described below. To help make the description of the behaviour zones easier to understand, they are described in reverse order (i.e. 4-1).

When the lamination thickness is on the order of the excavation radius, then an equivalent isotropic rock mass is found to satisfactorily represent the laminated rock mass. In Zone 4 of Figure 4.15, the non-jointed model crown deflections are similar to those of the jointed model. Also, the plastic yield zones, as shown in the model results of Figure 4.12, are similar between the two model types. There is some truncation of the yield zone in the laminated model.
at the haunch area. The specific location of the lamination with respect to the tunnel will control
the extent of plastic yield, and rock support should target these specific laminations.

As the lamination thickness decreases, the stresses begin to channel through the crown
beam, which begins to occur at a thickness of 7250 mm (Figure 4.15). The stress channeling
increases the stress through the crown beam causing increased yield and deflections, as seen in
Zone 3 of Figure 4.15. Again the plastic yield zone shape is controlled by the specific location of
the lamination. Note the shear failure underneath the lamination, cutting through the haunch area
(Figure 4.12, 3600 mm thickness), indicating that lamination slip is beginning to play an
important role in the excavation stability. Slip on the joint above the tunnel crown channels the
stresses into the crown beam. This is caused by dilation (normal displacement) of the lamination
above the crown, opening a void space, which forces the stress flow around the excavation
through the crown beam. As the lamination thickness decreases, a second lamination above the
crown begins to slip, causing the stress flow to be concentrated across two beams above the
tunnel crown. Below a lamination thickness of 2400 mm (Figure 4.15) the shedding of stresses
from one beam to the next is not as obvious, as in the stress channeling zone. In the modelling
presented here, the distance up from the crown to the first lamination has been fixed at half the
lamination thickness, therefore laminations within 3 – 4 m above the crown of the excavation can
begin to channel the stresses and increase the crown deflections. Rock support should continue to
target specific laminations to minimize slip so that the stress can be re-distributed into the
surrounding rock mass.

Clearly zone 3, Figure 4.15, indicates the start of deviations from the isotropic plastic
(rock plastic, no joints) model crown deflections. Considering the maximum plastic yield height,
measured from the center of the excavation vertically, there also appears to be a reasonable
correlation between the take-off point for plastic yield growth and the lamination thickness. The dashed lines in Figure 5.1 represent height (H) of half the lamination thickness (T/2) and 2/3 the lamination thickness (2T/3). As the plastic yielding begins to transition from the crown beam (T/2) and into the second beam above the excavation, the plastic yield height begins to increase. This also marks the transition from the stress channeling, zone 3 of Figure 4.15 into the multi-beam coupling zone, 2. This transition (~2400 mm) represents a key lamination thickness for engineering considerations.

![Figure 5.1: Maximum beam plastic yield height for various lamination thicknesses and rock mass properties at Ko = 3 and 150 m depth for 16 m diameter tunnels.](image)

\[ E_i = 4 \text{ GPa} \]
\[ E_i = 6 \text{ GPa} \]
\[ E_i = 8 \text{ GPa} \]
\[ E_i = 10 \text{ GPa} \]
\[ E_i = 20 \text{ GPa} \]

\[ H = T/2 \]
\[ H = 2T/3 \]
Above this thickness, rock support should target specific laminations to prevent lateral slip (Pells 2002) and below this thickness rock support for large plastic yield zones, discussed further in Chapter 6, should be considered. It should be noted that these are only preliminary recommendations and that extensive testing and modeling has not been conducted to validate these recommendations.

At a lamination thickness of 2400 mm, a second lamination unit becomes involved in the deformation process, and stresses are being shed to multiple beams causing plastic yielding and increasing the deflection. Below 1000 mm, the stresses are distributed throughout the rock mass, and are no longer channeled through one or two beams, and the crown deflections follow a similar trend as predicted by the voussoir model (Diederichs & Kaiser, 1999). The deviation from the classic voussoir model, which is plotted in Figure 4.15 as the dashed line, is due to the high stresses and the circular excavation geometry. The voussoir analogue is specific to gravity induced relaxation, although small amounts of confining stress can be incorporated into the iterative calculations (Diederichs & Kaiser, 1999). Zone 2 of Figure 4.15 also marks the start of tensile failure in the haunch area around the excavation. Plastic yielding and lamination slip now extend several meters above the excavation and are not truncated by the presence of the laminations, as in the stress channeling section. This results in further increases in the crown deflections and plastic yield height. The shape of the yield zone has near vertical sides above the crown, and joint slip extends well beyond the excavation (Figure 4.12). Rock support should now focus on tying the laminations together, to create a composite beam as first suggested by Lang et al. (1979) for support in long wall coal mining operations.

In Zone 2, to the left of the voussoir collapse threshold, of Figure 4.15, the degree of tensile failure has increased and extended into the laminations above the crown. This is indicative
of crown beam failure and the voussoir predicts snap-through of the laminated beam at 900 mm. However the circular excavation geometry provides additional support that is not accounted for by the voussoir model and the crown deflections do not increase as rapidly as the voussoir predicts. There is also more confinement and clamping provided by the high horizontal stresses than the voussoir analogue. The geometry of the plastic yield zone in Zone 2 is characterized by sub vertical sides and 'wings' which extend out from the haunch area. A high stress confinement area between the crown yield area and the haunch yield area, as indicated in Figure 4.12, exists. This high confinement area is sensitive to stress changes induced by rock fall or nearby excavation advance, which will redistribute the stress field and may result in further plastic yielding. Rock bolts installed in the high confinement area will have higher performance capacity than those installed in the plastic yield zone due to better anchorage. Resin grouted bolts, such as rebar, will not reach their full capacity if the plastic yielding creates open fractures, which will occur in TBM driven tunnels, if support is delayed.

At 420 mm, in Figure 4.15, the extent of plastic yield begins to stabilize in the finite element models and reducing the lamination thickness only has an elastic effect on the crown deflections. Note the slope of the anisotropic plastic model in Zone 1 of Figure 4.15 and the slope of the anisotropic elastic model. The degree of plastic yielding has damaged the rock mass in the yield zone to a point which has created a new elastic material.

The anisotropic behaviour, discussed above, appears to be independent of the tunnel radius. Various radii, from 3 to 8 m, were tested for rock properties associated with $E_i = 4$ GPa and the normalized plot of lamination thickness to vertical crown deflection, Figure 5.2, indicates good correlation with the anisotropic sections discussed above for all radii. The outliers can be explained because of the high mesh sensitivity.
Figure 5.2: Normalized vertical crown deflections showing consistent trends independent of radii for models with rock mass properties associated with $E_i = 4$ GPa at 150 m depth with $K_o = 3$.

When the zones of the mesh are slightly different dimensions at different radii, then the beam stiffness will not be exactly the same, creating differences in the relative vertical crown deflections.

There also appears to be good correlation of the behavioural sections, with respect to lamination thicknesses, at the various rock mass properties, defined in Table 4.2, as indicated in Figure 5.3 where the behavioural zones are labeled numerically. The behavioural zones are the same as those indicated in Figure 4.15.
Figure 5.3: Close up of Ko = 3 graph for various rock mass properties with behavioural zones indicated as 1) Plastic yield stabilization 2) Multi-beam coupling 3) Stress channeling through single beam and 4) Isotropic behaviour. Model results are for 16 m diameter tunnels at 150 m depth.\textsuperscript{6}

Changing the stress regime by adjusting the Ko ratio to 1 and 2 shows that as the stresses decrease, there is less anisotropic influence. This is shown in Figure 5.4 where the zones of anisotropic behaviour are only definable at Ko = 2 and at Ko = 1. The plastic yielding around the model excavations is insignificant and vertical crown deflections are essentially elastic deflections. This indicates that the anisotropic behaviour is stress sensitive, which agrees with findings of Chappell (1989).

\textsuperscript{6} This was presented in poster format by Perras & Diederichs 2009b and is included in Appendix B.2
Figure 5.4: Crown deflections for various rock mass properties at Ko ratios of 1 and 2. The zones of behaviour are numbered as 1) Plastic yield self limiting, 2) Multi-beam coupling, 3) Stress channeling, and 4) Isotropic.
5.1 Anisotropic Stress Dependency

The magnitude of the stresses around the excavation is an important factor controlling the magnitude of vertical crown deflections. To analyze this effect, models were computed at depths of 25, 75, 150 and 300 m, holding the Ko ratio at 3, and also models were computed at depths of 300 and 450 m with a Ko ratio of 1, which would result in the equivalent horizontal stress of a tunnel at 150 m depth with Ko ratios of 2 and 3, respectively.

Testing depths of 25, 75, 150 and 300 m indicates that the anisotropic behaviour still exists at all depths tested, however not all zones of behaviour are visible in the range of laminations tested, as shown in Figure 5.5. For example, the plastic yield stabilization zone at 300 m depth has not started to develop at 160 mm lamination thickness, and at shallow excavation depths (25 m), the zones are difficult to delineate without viewing that data set specifically, see inset graph of Figure 5.5. The inset graph, Figure 5.5, indicates a new behaviour style at large lamination thicknesses which causes crown inflection. This inflection is the result of reduced overburden confinement at the shallow depths with high horizontal stress, and the stiffer, thicker crown beams. The behavioural zones are not consistently defined at different depths with respect to lamination thickness, when the Ko ratio is held constant.

The stress ratio (Ko) has an impact on the confining stress acting on the rock mass around the excavation. By comparing model runs with equivalent horizontal stresses (i.e. \(\sigma_{ht} = 12\) MPa at 150 m depth with a Ko = 3 and at 450 m depth with a Ko = 1) it can be seen in Figure 5.6 that when the Ko ratio is greater than 1, then the vertical crown deflections are reduced compared to the equivalent horizontal stress at Ko = 1.

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7 Information from this sub-section was presented by Perras & Diederichs 2009a and is included in Appendix B.3
This demonstrates the effect of confining stress resulting from the opening of the underground excavations where the stress ratio is high, which creates high stress concentrations in the crown and invert focusing the plastic yielding at these locations. When the stress ratio is around 1, then the plastic yielding is more widely distributed around the excavation surface, creating a larger zone of vertical crown deflection due to increased tensile yielding in the haunch area, as illustrated in the model results of Figure 5.7.
Both the magnitude and the stress ratio influence the anisotropic behaviour of a horizontally laminated rock mass through the degree of plastic yielding, particularly the tensile failure, around the excavation. The behaviour at equivalent horizontal stress levels is not similar. Accounting for the magnitude and stress ratio in an anisotropic behavioural chart will be an important consideration, which will be discussed later.

![Diagram](image)

Figure 5.6: A comparison between increasing Ko ratios and equivalent depths to give rise to the same horizontal stress for models with rock mass properties associated with $E_i = 6$ GPa. Inset model results comparing the plastic yield zones.
5.2 Lamination and Vertical Joint Properties

The laminations play an important role in the deformation process. It has been shown that when the laminations are only accounted for through a classification system, that the deformations (yield and deflections) are under-predicted. To demonstrate the importance of lamination properties a series of models were computed with various Mohr-Coulomb lamination properties, as listed in Table 4.2. The tensile strength of the horizontal laminations has negligible effect on the crown deflections, as seen by comparing the tensile strength runs of 0.38 and 0 MPa, both with cohesive strength of 0.29 MPa. By adding random vertical jointing and maintaining 0 MPa tensile strength, there is an increase in the crown deflections. The trend is no longer a smooth curve, because localized rock yielding at the tips of the vertical joints is occurring.
The location of the vertical joints is held constant, however the location within the rock beams varies as a function of the lamination thickness and this causes irregular crown deflection for the series of model runs with the same strength properties. When the cohesive strength of both the horizontal and vertical laminations is reduced to 0.029 MPa, increased deflections occur, as expected. Again the deflections are a function of localized rock yielding at the vertical joint tips causing deviation from a smooth curve; however it is clear that lower lamination and vertical joint properties increase the vertical crown deflections.

Figure 5.8: Crown deflections for changing non-jointed models, horizontally laminated models and horizontally laminated with vertical joints models. Inset figure shows tensile failure (o) clustered locally around vertical joint tips.
The lamination and vertical joint properties have a strong influence on the magnitude of the vertical crown deflection. The most important parameters are cohesion and friction as indicated in Figure 5.8 and the least important is tensile strength. Defining lamination and/or joint properties for real projects can be a difficult task and is often done using classification systems or is based on precedent experience. Measuring the joint properties is impractical for engineering projects as it would require in-situ or large bulk samples. Sensitivity analysis is generally conducted to determine the influence of the joint properties on the stability of the excavation and then the appropriate conservative values selected for final design. Further research is necessary to rigorously test lamination and joint properties for horizontally laminated ground and to determine the effect on the stability of the excavation.

### 5.3 Anisotropic Failure Modes

Large excavation diameters for road and rail, water diversion tunnels, and other high capacity infrastructure tunnels are easily achievable with TBM technology and are in the order of 4 to 8m in radius. Projects of various dimensions (section 2.1.5) show that the horizontally laminated nature of the rock mass can cause increased notch or chimney style failure in the crown area due to lateral slip and bed deflection.

The vertical crown deflections were used as an indicator to determine when isotropic models would suitably represent an anisotropic material and the maximum plastic beam yield height was used as an indicator to determine the anisotropic failure mode. Taking this into consideration and the previous finding that the anisotropic vertical crown deflections begin to deviate from the isotropic response at a normalized lamination thickness of 0.9, a graph, (Figure 5.9), was created to determine the relationship between the model data and the failure modes.
The data results of this study were compiled into one graph, normalizing the x-axis to the tunnel radius and the y-axis to intact modulus. In Figure 5.9, all the numerical modeling runs are plotted and coded by the failure mechanism identified by the model results. The limits of the failure modes are also identified and plotted in Figure 5.9 to show how they fit the data. The failure mechanisms identified in the model results are;

1. Localized instability
2. Crown beam failure
3. Chimney failure

Figure 5.9 shows good correlations between the failure modes and the data, with the exception of the $E_i = 20$ GPa model results for chimney and beam failure. These points plot within the local haunch instability region, very close to the beam failure limit. Examining the model results for these points shows that the level of plastic yielding is minimal and the maximum shear strain is in fact localized within the crown beam as indicated in Figure 5.10. These model points could be classified as localized instability and in reality these model points would be stable, with only minor visible damage, such as induced cracking, apparent. Figure 5.9 is re-plotted in Figure 5.11 without the raw data and the failure modes are clearly identified.
Figure 5.9: All numerical modeling data results, coded by failure mechanism, to determine behavioural relationships. Limits of behaviour identified to show how they fit the data.
Figure 5.10: Model results for $E_i = 20$ GPa at a lamination thickness of 280 mm with a Ko ratio of 3 and a tunnel diameter of 16 m.
The axes of the graph are based on the previously mentioned fact that the ratio of lamination thickness to tunnel radius had no influence on the anisotropic behaviour for radii between 3 and 8 m and that the anisotropic behaviour is influenced by both the stress levels and the modulus of the rock. By creating a ratio between the stress level and modulus, this influence has been captured.

As indicated previously, plastic yield heights begin to rapidly increase when the plastic yield height is greater than the crown beam thickness or half the lamination thickness. This is a transition towards chimney style failure and the lower limit of T/2 was selected to be a conservative indicator.
The chimney failure mode is the result of large lateral slip on the laminations, resulting in deflection of the rock beams. The deflections induce plastic yielding, which starts in the haunch area and progresses out laterally and vertically. As the weight of the failing rock mass increases, tensile failure of the rock beam surrounding the excavation increases, and this causes progressive failure away from the excavation. The lateral slip above the crown confines the yield zone, resulting in a steep sided chimney.

The depth of plastic yielding has been contoured, for the chimney failure mode, in Figure 5.12. The chimney yield height increases as $\sigma_{\text{H}} / E_i$ increases and $T/R$ decreases. These contours
are based on the numerical models, and there is limited data, so coarse depth contours have been plotted and these should be used with extreme caution until refined with more data in the future.

Crown beam failure occurs when the plastic yielding is contained within the crown beam due to stress channeling, as discussed earlier, and little to no plastic yielding occurs in the beam above the crown beam. Increasing damage around the excavation can be expected as the chimney-crown beam divide is approached and conversely less damage towards the crown beam – haunch divide.

Localized haunch instability occurs due to low confinement in the haunch area and is enhanced when the tensile strength of the intact beam is low. The low confinement zone has been illustrated well by Bewick & Kaiser (2009). Haunch failure can be quite variable for both the model results, and in reality, and does not necessarily mean rock fall out and overbreak.

The gravity driven unraveling zone was determined using the voussoir analogue and by applying a minimum amount of lateral stress to initiate beam failure. It is only included for completeness, and a detailed study of gravity driven unraveling failure is beyond the scope of this work.

The anisotropic failure mechanisms described above are based on numerical modeling methods, and as such certain limitations exist, which will be explained in the following section.

### 5.4 Limitations of the Anisotropic Plasticity Method

Certain assumptions and limitations in the model parameters had to be made to limit the scope of this research. These limitations have a crucial impact on the usage of the information presented and caution should be used as this is a preliminary study of tunneling in horizontally laminated ground. The limitations include:
• intact lamination properties
• limited study of vertical structural influences
• synthetic intact modulii between 4 and 20 GPa loosely based on sedimentary rock properties
• shallow excavation depths with Ko ratios between 1 and 3
• tunnel radii between 3 and 8 m
• continuum mechanics solution scheme

The intact lamination properties have a major impact on both the vertical crown deflections and the depth of plastic yielding. To limit the number of model runs it was decided to only model “intact” planes of weakness, which have strength parameters close to that of the intact rock. True open fractures and joints would have strength values much less than the intact rock. The results of this study should be used for anisotropic rock masses which feature intact bedding and layering, with widely to very widely spaced joints.

The normal and shear stiffnesses of the laminations are critical in the deformation process. This study used a normal to shear stiffness ratio, \(K_n/K_s\), of 10. Other ratios were beyond the scope of this study. However, it should be noted that as the \(K_n/K_s\) ratio decreases the system becomes stiffer. This increase in stiffness will result in less plastic yielding at the same stress level and essentially raises the chimney – crown beam failure divide. The crown beam – haunch instability divide is not affected, as severely, by the stiffness of the laminations. This
division, crown beam – haunch instability, is influenced mostly by the stiffness of the rock beams.

A very limited study of the effect of vertical jointing on the anisotropic behaviour was carried out at a Ko ratio of 1, such that stress clamping effects would be minimized. The results indicate that the vertical jointing causes increased deflections and that localized tensile yielding at the joint tips causes minor irregular behaviour of the model. The full extent of these irregularities due to the vertical joints was not fully explored in this research and as such the results should be used with caution where vertical structure is present.

The rock mass properties were derived based on the author’s experience with sedimentary rocks of Southern Ontario and the range of values were intended to cover sedimentary rocks such as sandstones, siltstones and shales. The values were selected to progressively increase in strength. Anisotropic rock masses can include all the major rock types, and better derivation of specific rock mass properties which represent an average shale, for example, should be part of a further study on the anisotropic behaviour. This would allow ranking of particular rock types, according to their anisotropic behaviour, more precisely.

A range of excavation depths and stress ratios were tested, as both were found to have significant influence on the anisotropic behaviour. The depths tested should be considered to be shallow (< 450 m) and it has been suggested by Amadei (1996) that at great depth the anisotropic nature of a rock mass is much less pronounced because of the high normal to the plane stress. This effect was not observed in the model results presented here, due to the shallow depths examined. Similarly the stress ratio creates confinement which appears to have an inverse effect on the plastic yield zone. When the Ko ratio is low, the stress flow around the excavation does not create large confinement due to stress concentration. As the stress ratio is increased, the
stress is concentrated on the excavation surfaces which parallel the high stress alignment. When
the laminations of the rock mass and the high stresses are parallel, the stress flow path around the
excavation is altered, as the stresses preferentially flow through the rock beams. From this
understanding it was decided not to test Ko ratios of less than 1. In horizontally laminated ground
where the maximum stress is vertical, a low confinement zone above and below the excavation
will develop. Tensile stresses could develop in the low confinement zone under the right
circumstances. If tensile failure were to develop it would fracture the rock beams and cause
overbreak. This failure process has not been identified on the anisotropic failure mode plot.

The majority of the modeling was conducted with Phase2, a finite element continuum
code, and only selected runs were compared with the results of UDEC, a distinct element
discontinuum code. The UDEC results did indicate good agreement with the Phase2 plastic yield
zone size and shape at the different stress regimes, as indicated by comparing Figure 4.12 and
Figure 4.13. Without the inclusion of vertical structure in the model runs, UDEC is unable to
fracture the rock beams and cause detachment and overbreak. The rock beams can simply slip
past each other and deform internally. This results in the mechanistic pre-yield behaviour, but
once yielded, the rock beams continue to deflect and are not allowed to break apart. This
limitation could be overcome by applying vertical structure which would allow beam break up,
however the overbreak geometry would be highly dependent on the structural spacing. Further
testing is necessary to determine the effects of vertical structure on the modeling results.
5.5 Overbreak Assessment Using Data from the Niagara Tunnel Project

Applying the anisotropic failure mode chart, given in Figure 5.11, to the formations of the Niagara Region further enhances the modeling results. Good agreement is shown between the observed deformations from the Niagara Tunnel Project and the location of the formations plotted on the anisotropic failure mode graph, as shown in Figure 5.13. The data is included in Table 5.1.

Figure 5.13: Formations from the Niagara Tunnel Project plotted on the anisotropic failure mode chart. Formation properties based on data from the Niagara Tunnel Project (courtesy of Ontario Power Generation) and the authors observations.
The minimum and maximum $\sigma_{H}/E_i$ values are used to cover the range of stress and strength properties within each formation. The lamination thicknesses were derived from field observations, first based on major sedimentary layering within the formation, and where overbreak was known to occur, based on an average slab thickness. The anisotropic failure mode graph predicts chimney and crown beam style failure of the Rochester, Grimsby and Queenston Formations, with potential localized instability at the lower end of the spectrum.

Table 5.1: Data from the Niagara Tunnel Project (courtesy of Ontario Power Generation, with the exception of the Thicknesses which are based on the author’s observations).

<table>
<thead>
<tr>
<th>Formation</th>
<th>$\sigma_{H}$ (MPa)</th>
<th>$E_i$ (MPa)</th>
<th>$\sigma_{H}/E_i$</th>
<th>$\sigma_{H}/E_i$</th>
<th>Thickness (mm)</th>
<th>Thick-ness / Radius</th>
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<td>150</td>
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<td>0.00032</td>
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</tr>
<tr>
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<td>16000</td>
<td>0.00033</td>
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Observed overbreak in the Rochester Formation was in the order of 0.5 to 1.0 m deep and the notch was 3 - 4 m wide on either side of the center line, with stepped sides, as shown in Figure 3.17. Slabs, in the Rochester, Power Glen and Queenston, were generally of in the order of 150 mm thickness and for clarity 145 mm, for the Rochester, was used for plotting purposes to clearly distinguish the Rochester from the Queenston. The depth of failure predicted from Figure 5.13 ranges from 0, localized haunch instability and shallow crown beam failure, to ~1.5 m chimney style failure. This is in close agreement with the observations from the tunnel. A typical overbreak profile is presented in Figure 5.14, with an inset photograph from the tunnel, showing the stepped sides of the profile caused by bed parallel fracturing. This could be considered to be a shallow chimney style failure at the margin of crown-beam and chimney failure modes.

![Overbreak profile from the Niagara Tunnel Project for the Rochester Formation. Inset photo shows bedding plane parallel slabs and stepped profile (data and photo courtesy of Ontario Power Generation).](image)

Figure 5.14: Overbreak profile from the Niagara Tunnel Project for the Rochester Formation. Inset photo shows bedding plane parallel slabs and stepped profile (data and photo courtesy of Ontario Power Generation).
The Neagha Formation, although not visible in Figure 5.14 due to the very small lamination thickness, is predicted to result in chimney failure around 3 m deep. The observations from the Niagara Tunnel would suggest that the failure process was gravity driven, due to the fissile nature of the rock mass. This formation broke back to the overlying formation consistently.

Only minor crown beam failure and localized haunch instability were observed in the Grimsby Formation at the Niagara Tunnel Project. The characteristic failure mode resulted from induced fracturing along shale – sandstone bedding interfaces and localized fallout along shale beds, which were on average 100 to 200 mm thick. The high end of the spectrum is based on the shale layers and is plotted to show the influence of the interbedded nature of the Grimsby.

Based on Figure 5.13, significant chimney style failure could occur if the shale layers controlled the failure process, however without additional discontinuities (i.e. vertical joints) the sandstone layers controlled the stability in the Niagara Tunnel (sandstone layers were >0.5 m in thickness, shale < 0.2 m), and as predicted by Figure 5.13 only localized instability occurred. The Power Glen Formation shows localized instability in Figure 5.13, and closely plots near the gravity driven failure mode limit. Overbreak observed in the Power Glen Formation was typically limited to the haunch area locally for the upper unit (mostly sandstone) and shallow depths of overbreak above the crown elevation resulted in the lower unit, as shown in Figure 5.15. The Power Glen shale is in fact weaker than the Queenston, when comparing UCS values, and the stress levels are lower. This combination has resulted in minor stress induced, gravity assisted failure, of the lower Power Glen unit.

Overbreak in the Queenston can be broken down into several categories, the Whirlpool-Queenston contact area, the approach toward St. Davids Buried Gorge, St. Davids Buried Gorge
zone of influence and high horizontal stress field after St. Davids Buried Gorge. Typical overbreak profiles for each category are presented in the following figures (5.16 to 5.19) showing the range of observed behaviour within the Queenston.

Figure 5.15: Overbreak in the lower Power Glen Formation restricted by more competent units above and below, creating localized haunch instability (photo courtesy of Ontario Power Generation).
Figure 5.16: Typical overbreak profile for the Whirlpool-Queenston contact area. Inset photograph shows the Queenston broken away from the Whirlpool (data and photo courtesy of Ontario Power Generation).

Figure 5.17: Typical overbreak profile for the approach to St. Davids Buried Gorge. Inset photograph showing typical overbreak in the order of 0.5 – 1.0 m deep (data and photo courtesy of Ontario Power Generation).
Figure 5.18: Typical overbreak profile for the St. Davids Buried Gorge influence zone, prior to spile installation. Inset photo shows (data and photo courtesy of Ontario Power Generation).

Figure 5.19: Typical overbreak profile for the high horizontal stress field after St. Davids Buried Gorge. Inset photo showing overbreak up to ~ 3 m deep (data and photo courtesy of Ontario Power Generation).
Figure 5.14 predicts chimney failure, crown beam failure and localized instability for the Queenston. Based on the observations above, chimney and crown beam failure modes dominate. The lower end of the predicted mode of failure for the Queenston is based on a low stress measurement, which comes from near the Whirlpool-Queenston contact. Despite the low stress levels, gravity driven unraveling is not predicted. If the low stress level is increased to 9 MPa, from 5.3 MPa, then only chimney and crown beam failure modes are predicted, which agrees with observed overbreak from the Niagara Tunnel Project, however the depth of plastic yielding does not agree with the depth of overbreak. Generally speaking, numerical plastic yielding does not mean rock fall out and overbreak. At low lamination thicknesses, the anisotropic behaviour suggests that a self stabilizing geometry can be achieved, and it is possible that although extensively yielded, rock mass hang up could be occurring, or support is installed to minimize the depth of overbreak at the Niagara Tunnel Project. The influence of rock support has not been included in this study, and the influence of the excavation method and support installation timing is a recommended area of future study.

The remaining formations encountered by the Niagara Tunnel fall in the localized haunch instability region or at the boundary of gravity driven unraveling. This agrees well with the observations from the tunnel as most of these units were stable and only localized fracture growth at bedding planes occurred and dilation within the haunch area could be observed. When thick shale beds closed in the crown, local fall out of the shale due to self-weight occurred. Photos of the various formations, where localized instability or stable conditions exist, are shown in Figure 5.20 and Figure 5.21.

The new anisotropic plasticity method was used to develop a graph for predicting the failure modes associated with anisotropic behaviour. Good agreement between observations and
measurements from the Niagara Tunnel Project and the predicted failure modes in Figure 5.13 has been achieved. With refinement this tool could be used to determine overbreak depths of unsupported excavations and to determine the required support.

Figure 5.20: Formations from the Niagara Tunnel Project showing localized instability issues or stable conditions. A – Grimsby shale layers closing in the crown and B – stable Whirlpool sandstone (photos courtesy of Ontario Power Generation).
Figure 5.21: Formations from the Niagara Tunnel Project showing localized instability issues or stable conditions. A – Lockport with shale parting, B – Lockport at portal, C – Irondequoit, D – Reynales with shale parting closing in crown and E – Thorold with shale interbed (photos courtesy of Ontario Power Generation).
Chapter 6: Excavation Design Recommendations

6.1 Excavation Shapes – Hydraulics versus Stability

To determine the appropriate tunnel dimension for a hydropower diversion tunnel the sum of the construction costs, maintenance cost and head loss cost (i.e. power not produced due to head losses) must be minimized (Gulliver & Arndt, 1991). Fahlbusch (1988) studied 394 concrete and steel lined penstocks and found that the economic diameter for concrete lined conduits can be found using:

\[ D = 0.62Q^{0.48} \]  \hspace{1cm} (6.1)

with +/- 20 % accuracy. This provides a preliminary starting point for hydro power tunnel design. This empirical relationship has been plotted in Figure 6.1 and is for circular tunnels. The equation is not site specific and as such should be used with great caution, as the geological conditions may not support the optimum financial diameter. Under these situations further benefits could be realized by choosing a non-circular cross section or two tunnels to give the equivalent cross sectional area.
Traditional tunnel construction, using drill and blast methods, often opted for horseshoe shaped tunnels as benefits were realized during construction with a flat invert. With the increasing use of TBMs, a circular tunnel is more frequently specified. This is the optimal shape because the excavated cross-sectional area is the smallest for a circular shape, for a given hydraulic radius.
If the hydraulic radius remains constant, then the frictional head losses due to wall roughness will also remain constant for various tunnel shapes, as per the frictional coefficient ($f$) and the head loss due to friction ($H_f$) equations (6.2 (Giles, 1962) and (6.3 (Gulliver & Arndt, 1991));

$$f = \left( \frac{1}{4 \log \left( \frac{Dh}{\epsilon} + 1.74 \right)} \right)^2$$  \hspace{1cm} (6.2)

$$H_f = f \frac{L V^2}{Dh 2g}$$  \hspace{1cm} (6.3)

$$Dh = 4 Rh$$  \hspace{1cm} (6.4)

These values are dependent on the hydraulic diameter ($Dh$) when all other parameters remain constant for comparison, where $\epsilon$ is roughness, $L$ is length of tunnel, $V$ is mean flow velocity and $g$ is gravitational acceleration. Therefore the tunnel shape is independent of the hydraulic properties and is a function of the stability of the rock mass.

To determine the most stable tunnel shape in horizontally laminated ground, three additional, non-circular models were computed and compared to the circular case with lamination thickness of 280 mm, hydraulic radius of 4 m, at 150 m depth with a $K_o = 3$. The three excavation shapes are shown in Figure 6.2 with contours of maximum shear stresses around the excavation and plastic yield indicators.
Figure 6.2: Comparison of stability of non-circular shaped excavations with the same hydraulic radii \((R_h)\), which is calculated by \((\Lambda)\) area / \((P)\) perimeter, as a 16 m diameter circular excavation. Maximum shear stress contours are shown, and plastic yield limits were used in conjunction with vertical crown deflection \((\Delta)\) to determine stability.

The model results show that the ellipse is the most stable cross sectional shape having the lowest vertical crown deflection and the narrowest and shortest plastic yield zone. The ellipse
could result in more wall instability difficulties than the circular cross section, however, as the elliptical shape does not shed the high horizontal stresses as effectively. However, experience at the Niagara Tunnel suggests that in horizontally laminated ground wall, instability is a minor issue.

The elliptical shape would be the hardest to excavate and would most likely have to be excavated by top heading and bench due to the extreme height of the excavation (20 m). This drawback would out way the benefits achieved by this shape in terms of stability of a tunnel, however for caverns the elliptical shape may be of some benefit as it minimizes the span of the excavation, thereby reducing the plastic yield zone.

The horseshoe shape has no advantage over the circular cross section, since the crown dimensions are the same. There is also increased sidewall yielding due to the flat corners at the invert which results in a sharp stress concentration. The benefit of the horseshoe shape would be realized when tunneling with drill and blast, or road header methods, since a flat invert is more desirable for traffic. However at such large diameters, this benefit becomes less of an issue due to the relative flat curvature of the tunnel floor.

The square cross section is the least desirable shape as it has the largest plastic yield zone, both laterally and vertically; however the vertical crown deflections are similar to the circular and horseshoe cross sections. Pells et al. (2002) suggested that a flat crown is the optimum shape for horizontally laminated ground at low stress ratios (<2). The square cross section tested at Ko=3 does not agree with Pells et al. (2002), however the test case is unsupported. If support were to be simulated, then the plastic yield zone should improve in the square cross section, as with all cross sections.
To achieve the various shapes which are non-circular, drill and blast, or road header excavation methods must be employed. It is possible to modify a TBM such that elliptical cross sections or dual circular sections are achieved; however as yet there have been few of these machines put into practice for hard rock tunnels. The excavation method will impact the stability of the excavation in horizontally laminated ground, and certain benefits can be utilized with each method.

6.2 Excavation Methods

The excavation method will have an impact on the stability of the underground opening, and benefits for each method can be realized which will increase the stability and productivity of tunneling in horizontally laminated ground. All excavation methods used in horizontally laminated ground should minimize the distance between the face and installation of rock support. This will reduce the depth of overbreak if chimney style failure is predicted to occur. This failure mode is a gradual failure process as the stress field re-establishes itself around the excavation.

For drill and blast excavations this distance is controlled by the length of each blast round. The timing of the installation is critical and efforts should be made to ventilate the face as quickly as possible so that rock support installation can begin. With TBM excavation, the distance between the face and support installation is controlled by the cutterhead dimensions. For horizontally laminated ground, the cutterhead dimensions should be minimized and a stiff roof shield used to minimize rock movements prior to support installation. Modification of the traditional TBM cutterhead arrangement may be necessary for large diameter TBMs so the cutterhead dimensions can be minimized. These modifications could include motor re-
arrangement, telescopic working platforms, re-routing hydraulic and electrical lines and inclination of the cutterhead. These modifications could decrease the cutterhead length, increase the amount of working space at the front of the TBM and prevent critical components from being damaged by rock fall. By reducing the cutterhead length, rock support can be installed earlier and this could reduce the volume of overbreak.

Roof support prior to the installation of rock support can be achieved with jacks placed on the cutterhead of a TBM so as to raise a plate into contact with the rock mass. Many TBMs utilize sidewall jacks to stabilize the machine during excavation and prevent eccentric rotations. These jacks also help steer the machine. Jacks at the crown, although sometimes used, may not be capable of producing enough pressure to stabilize the rock mass or may not have the required extension if minor overbreak is already occurring. Establishing a pressure on the rock will reduce the amount of deformation experienced when rock support installation is possible.

When using drill and blast methods, the primary plane of weakness, the horizontal laminations, can be utilized to establish a pre-split line so that damage to the surrounding rock mass is reduced. By drilling periphery holes parallel to the laminations, where possible, a well established and neat pre-split line can be achieved. This is only possible when the laminated nature of the rock mass is such that it preferentially splits along the laminations. This method was in fact was used in the 1950’s hydro tunnels at Niagara, as mentioned earlier.

When tunneling in horizontally laminated ground, utilizing the horizontal nature of the rock mass can improve the excavation rates and reduce cost associated with overbreak. Modifications to the standard excavation methods may be necessary to optimize the performance of the excavation method in horizontally laminated ground.
6.3 Excavation Support in Horizontally Laminated Ground

It has been shown that the plastic yield zone around an excavation when modeled with discrete anisotropy, differs in size and shape from that modeled using traditional isotropic material methods. The material softening method (RocScience 2008) was used to simulate 3D tunnel advance in 2D by progressively reducing the modulus of the material inside the excavation. Inspecting the model results, Figure 6.3, for the softening method, shows that the height of the plastic yield zone, with Ko = 3, triples in the last stage of model softening.

As can be seen, in Figure 6.3, the increase in the plastic yield height for the isotropic material properties has already reached the maximum height at 10% softening. This difference in the development of the plastic yield zone as well as the difference in size and shape can be utilized for rock support installation.

With the early installation of primary rock support, such as bolts, channel and mesh, the plastic yield may be contained and potentially reduced. From a TBM, the rock bolts could be angled forward such that they apply a force to the rock mass (tensioned) ahead of the back of the cutterhead; see Figure 6.4. Including channels and mesh to contain the yielded rock will reduce the volume of overbreak significantly. For pressure tunnels, rock mass grouting will have to be conducted as plastic yielding will still develop and create fractures. The confinement from the rock support should reduce the dilation of the rock mass and thereby reduce the volume of grout required.
Figure 6.3: An example of the material softening method used to simulate 3D tunnel advance in 2D. a) Material inside tunnel at 10% of original $E_i$ value, and b) fully excavated. Model results are for a 16 m diameter tunnel with lamination thickness of 280 mm or equivalent isotropic properties at 150 m depth with $K_o = 3$. 

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Figure 6.4: Illustration of chimney style failure and rock bolt installation.

Adjusting the length and orientation of the rock bolts could result in increased efficiency in the support system. Bolts may need to be lengthened closer to the center line of the tunnel, so as to exceed the plastic yield zone. The orientation of the rock bolts is often governed by the drills for TBM excavations. Careful consideration should be given to the possible orientation of rock drills so they are maximized to control the geological conditions. With further modeling of rock support for horizontally laminated ground, a set of rock support recommendations could be developed to coincide with the anisotropic failure mode plot.
Chapter 7: Summary, Future Studies and Conclusions

7.1 Summary of Findings

Tunnelling in horizontally laminated ground presents some unique challenges. One of these challenges is that the primary structural feature follows a shallow tunnel drive for long distances. This will increase the probability of structurally induced failures (not dealt within this research). This research has shown that the inclusion of discrete anisotropy in numerical models modifies the predicted shape of the plastic yield zone, and increases the crown deflections as compared to those predicted by traditional isotropic material modelling. The current Niagara Tunnel Project has provided the backdrop for a larger, more general study of tunneling in horizontally laminated ground. To fully understand the yielding mechanisms of the Niagara Tunnel and apply them to a generalized modeling methodology for use with other ground conditions, a detailed understanding of the geological history of the Niagara Region, particularly the depositional environment, stress history and glacial impact on the topographic features, was necessary. Applying the geological history of the Niagara Region and practical observations from the Niagara Tunnel Project to the study of tunneling in horizontally laminated ground has led to the development of a practical tool for determining when anisotropic modeling analysis should be conducted and the anticipated failure mode. Further study is necessary to refine this tool for application over a wide spectrum of geological parameters and to test its accuracy with further case studies.
The Niagara Tunnel Project has descended down through the entire stratigraphy of the Niagara Escarpment, passed under St. Davids Buried Gorge and ascended back into the formations above the Queenston. The Niagara Tunnel Project has provided a unique opportunity to study the influence of a large diameter excavation on the behaviour of horizontally laminated sedimentary rocks of varying lithologies and properties. Overbreak occurred in some of the formations, most notably the Queenston Formation where depths of up to approximately 4 m were measured. The large overbreak depths were overcome by using spiling to advance the tunnel and later modifying the TBM to allow large volumes of rock fall out and increased mucking capacity at the invert. The Niagara Tunnel Project prompted a larger study into the response of horizontally laminated ground and the potential failure modes of various laminated rock masses.

To better understand the behaviour of the formations at Niagara, a detailed understanding of the geological history was necessary, from the depositional environment to glacial loading and unloading and neotectonic influences. The sediments have been derived from the Taconic orogenic belt along the east coast of North America. A large sedimentary basin developed and the formations around Niagara, although escaping orogenic deformations, acquired locked-in horizontal stresses. These horizontal stresses were increased due to glacial loading and once the ice loads were relieved, the horizontal stresses were locked-in causing horizontal to vertical stress ratios of 2 – 5 or more in Southern Ontario. The glacial impact also has had localized effects on the rocks of the Niagara Region. The Niagara Escarpment was in existence prior to the last glacial advance and the impact of the ice on this obstruction would have caused plastic yielding both at the escarpment edge and farther in land. Evidence exists to suggest that St. Davids Buried Gorge may have been eroded by high pressure melt water underneath a glacier, in a tunnel valley,
creating the truncated gorge alignment with an undulating and non-continuously falling thalweg. The Niagara Gorge may also have been initiated by glacial plucking or tunnel valley processes and further extended by river erosion. Despite the origin of these topographic features, their influence on the local stress field and the structural features of the Niagara Region are evident, and influence the behaviour of the rock mass.

Four behavioural responses and three failure modes due to large diameter excavations in horizontally laminated ground have been identified. When the lamination thickness is in the order of the excavation diameter, then the rock mass response is isotropic. The failure of the rock mass will be localized if it occurs and is focused in the haunch area of a circular excavation. As the lamination thickness decreases, stress channeling elevates the stresses in the crown beam, localizing plastic yield and increasing the vertical crown deflections until the stresses are shed to multiple beams at lower thicknesses. The stress channeling causes crown beam failure to occur and minimizes the depth of overbreak by concentrating the plastic yielding. Once the stresses begin to be shed to multiple beams, the vertical crown deflections decrease slightly. This marks the beginning of chimney style failure, where a steep sided notch develops above the tunnel. Multi-beam coupling behaviour is similar to the voussoir analogue; however the crown deflections vary from this analogue due to the circular nature of the excavation and the stresses. A stage is reached when decreasing the lamination thickness further has minor influence on the rock mass behaviour and failure mode. The laminations are sufficiently small that the depth and shape of the plastic yield zone is no longer controlled by the lamination thickness. However the shape of the plastic yield zone remains a function of the anisotropic nature of the rock mass. These behavioural responses and failure modes do not develop in the same manner if the rock
mass is modeled as an equivalent isotropic material. The equivalent isotropic method under predicts the vertical crown deflections and the depth of plastic yielding.

The work has been summarized in Figure 5.11 which represents a tool for predicting when anisotropy becomes an important consideration for numerical modeling and the failure modes expected in horizontally laminated ground for circular tunnels at shallow depth. The observations from the Niagara Tunnel Project have been applied to this tool and there appears to be good agreement between the observed failure modes and those predicted. Further refinement of this tool and the modeling process is necessary to develop a robust design chart for use in engineering design.

7.2 Recommended Future Studies

This research is a preliminary investigation into the behaviour of large diameter excavations in horizontally laminated ground. Although much research has been conducted on anisotropic strength of laboratory samples, the ability to numerically model excavation scale geometry with thin laminations has only been recently practical due to increasing computational power and software developments. It is now feasible to model an excavation in horizontally laminated ground for engineering design purposes using the discrete anisotropic plasticity model presented here.

Smaller scale models could be employed to both improve efficiencies of computational time and to model behaviour at lamination thicknesses below 160 mm. A half space or even quarter space model should be first compared to the full scale model results presented here to determine proper boundary conditions and ensure that the behaviour and failure mechanisms are
repeatable using the reduced model space. The reduced model space will decrease the demands on the numerical calculations and further refinement in the mesh element size should be conducted so as to reduce the mesh dependent stiffness of the rock beams. The reduced model space could also be utilized to test lamination thickness below 160 mm to better refine the mechanistic behaviour at low lamination thicknesses.

A rigorous testing of lamination and joint properties, as well as residual strength and dilation parameters is necessary to determine the influence on the anisotropic behaviour and further refine the failure mode limits. Back analysis of case histories will provide the necessary data, for comparison and prediction, and the use of residual strength and dilation will be necessary to accurately simulate the overbreak zone from various projects.

A detailed back analysis of the overbreak from the Niagara Tunnel Project was beyond the scope of this research, however this work is ongoing and will be presented at the upcoming International Tunnelling Association conference in 2010 (Perras et al., 2010). The back analysis will include: (i) empirical tools for tunnel damage prediction and conventional rock mass plasticity analysis, (ii) elastic analysis with progressive removal of damaged material, (iii) elastic-brittle, strain dependent – cohesion weakening and friction softening, and (iv) and the discretely anisotropic model which was developed during this research.

Rock support is necessary to maintain stability and safety in underground excavations. The next step would be to determine rock support requirements for the various failure modes identified and to determine the influence on the anisotropic behaviour for various tunneling methods. This work could develop numerically based, and case history proven, design charts for rock support selection when tunneling in horizontally laminated ground. This information could prove particularly useful to the nuclear industry as many organizations from around the world are
researching the storage of spent nuclear fuel in deep geological repositories (DGR) in sedimentary rocks. As this research is largely based on practical observations from the Niagara Tunnel Project it is of particular interest for Canadian nuclear repositories, as ongoing investigations in the northern end of the Appalachian sedimentary basin are being conducted.

7.3 Conclusions

The new discretely anisotropic method developed in this research demonstrates the importance of incorporating the anisotropic nature of a rock mass in numerical modeling. By utilizing joint elements to induce the mechanistic behaviour of an anisotropic material, a more realistic yield zone is established. It was determined that this mechanistic behaviour becomes an important consideration at a lamination thickness to radius ratio of 0.9. Below this threshold localized haunch instability, crown beam failure or chimney failure will result when there are nominal stress levels. Gravity induced unraveling will occur when the stress levels and modulus are low. The new discretely anisotropic method was used to develop a failure mode plot with contours of plastic yield depth. The modes of failure and depths of yield are in agreement with observations from the Niagara Tunnel Project and other case studies. Further case studies and refinements are necessary to develop this tool into a robust design chart for failure mode prediction and rock support recommendations.
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Appendix A: Swelling Behaviour of the Queenston Formation
Engineering Projects in Swelling Ground

A look at the process of investigation and design of engineering structures on and in swelling ground

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Prepared by - Matthew Perras
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Table A.2: Swell potentials for samples tested during January 30 to May 2, 2007.
A.1 Introduction

When constructing on and in swelling ground, special attention must be paid to the expansive nature of the surrounding rock mass during the investigation and design phases of the project, as well as during construction. During site investigation the collected samples must preserved so that the moisture content of the sample remains similar to that of the in-situ rock mass. Laboratory testing will better define the swelling process and determine swelling pressure which could be generated. The design must consider the ways to eliminate or absorb the swelling pressures generated on the structure and numerical modeling can be conducted to determine the rock mass and structure interaction behavior. During construction, water management has an important role in minimizing swell damage, which can lead to increased swelling during the life of the structure. Throughout the entire project life cycle, proper management of the swelling material must be conducted in order to minimize the swelling effects.

A.1.1 Projects in Swelling Ground

Many projects have been constructed on and in swelling ground. In Ottawa, Ontario, many foundations are constructed in black shale of the Billings Formation, which exhibits swelling behavior, and can lead to foundation cracking and heave if not properly considered in the design phase (Harper 1979). Tunnels in Europe have been constructed through anhydritic shale rocks, (Steiner 1993), which can swell when water transforms the anhydrate into gypsum. This swelling process results in extreme heave and crushing of reinforced invert arches (Steiner 1993). Research is being conducted into underground cavern storage of nuclear waste in swelling shale formations, so that the swelling nature can be used to seal fractures generated during construction. The current Niagara Tunnel Project is being excavated by a 14.4 meter tunnel boring machine...
(TBM) in Niagara Falls, Ontario. This project will pass through the Queenston Formation, which is a swelling siltstone. These projects have or will be constructed on and in different material which exhibits varying swelling behavior.

**A.1.2 Swelling Behavior**

Several swelling mechanisms of rocks, in general, have been reported by researchers and include hydration of anhydrite, double layer theory, pyrite oxidation and ionic diffusion. The first three are described briefly below for completeness and the fourth is described in more detail as it pertains to the Queenston Formation, used here in as an example.

The transformation of anhydrite, by hydration, into gypsum causes a volume increase and results in swelling of the rock mass. It has been noted that this process can result in a volume increase as much as 60%.

The swelling behaviour of clay minerals has been described using the double layer theory. The theory, which Olson and Mesri (1970) agreed accounts well for the swelling behaviour of clay minerals, implies that clay minerals are represented by two thin plates that are held together by water molecules. When the plates encounter a source of water, more water molecules can be absorbed in between the plates, causing expansion.

The black shale of the Billings Formation in and around Ottawa has caused basement heave in buildings of this region (Harper 1979). The heave results from a series of chemical reactions in which carbonates react with sulfuric acid formed from oxidation of pyrite, catalyzed by bacteria. This reaction forms crystals of jarosite \( 2\text{KFe}_{3}(\text{SO}_{4})_{2}(\text{OH})_{6} \) and gypsum \( \text{CaSO}_{4} . 2\text{H}_{2}\text{O} \), often in existing fractures and bedding plane partings (Harper 1979).
Swelling of the shale units in the Niagara region is well documented, and is reported in terms of ‘swell potential’, or the strain rate per log cycle of time in days. Based on laboratory measurements of time-dependent deformation by Dr. K.Y. Lo (1989) and others, it is found that horizontal swelling potential of the Queenston Formation is isotropic and that the vertical swelling potential is up to 1.6 times the horizontal swelling potential. The swelling deformation response is stress-dependent, and can be represented by a linear relationship between swelling potential and applied stress in a semi-log plot and that swelling can be completely suppressed under 4 to 5 MPa stress (Rigbey 2007).

The swelling process is associated with ionic diffusion of salts from the connate pore water in the rock, and a corresponding reduction in capillary and surface tension and fresh water uptake within the pores. Swelling is initiated by the relief of initial stresses to below the swell suppression pressure, accessibility to fresh water and an outward salt concentration gradient from the pore fluid of the rock to the ambient fluid (chloride diffusion), (Lo 1989). This process is a significant consideration for the tunnel design, as tunnel excavation, followed by the introduction of a significant fresh water source to the Queenston Formation would initiate swelling.

**A.1.3 Investigation through to Design**

This report is intended to outline some of the special needs of investigation through to design phases of construction projects on and in swelling rock masses, using the Niagara Tunnel Project as an example for the swelling process of the Queenston Formation. It is not intended as a complete guide for investigation through to design of geotechnical projects, although some aspects not related to swelling are mentioned for clarity.
A.1.4 Site Investigation

A.1.4.1 Geological Setting

The first phase of any Geotechnical engineering project is the desk top study. At this point it is necessary to determine the geological setting of the project and to collect already published information which can be used to plan the phase 2 and 3 investigation programs. This information often consists of regional geological maps, nearby site specific geological reports, aerial photography and remote sensing (Goodman 1993). A summary of the above information should lead to a general understanding of the geological history of the area and the types of rocks which can be expected on the construction site, their character and general rock mass behavior to the construction process. One should also be able to determine if there is potentially swelling rocks based on the geological history. A ground investigation program can be developed to answer some of the remaining questions which are necessary for the design of the structure and define more clearly swelling behaviour.

A.1.1.1 Ground Investigation Program

The ground investigation program or phase 2 investigations for a geotechnical engineering project will include borehole drilling to determine rock mass type, thicknesses, and quality. Down the hole tests can be conducted to determine permeability and stress field orientations.

The ground investigation program should be flexible enough so that it can be refined based on the initial findings. Detailed field mapping will identify weak units that weather easily, which result in talus slopes and are often caused by weak shale layers and high clay content. According to Einstein (1994b) high clay content indicates potential swelling and slaking
behavior. The commission on swelling rock (Einstein 1994b) suggests several easy tests which can be conducted on field samples to help identify clay mineral content.

Smear test: The residue of a wet argillaceous sample will feel soapy when the rubbed between fingers and sand on soap indicates some quartz or feldspar.

Taste test: The residue of a wet argillaceous sample will feel creamy in the mouth and a grinding powder feel indicates some quartz or feldspar. A bitter or salty taste suggests evaporite minerals. **Do not taste potentially contaminated samples.**

Water reaction test: A dried sample of argillaceous material will begin to deteriorate within 30 seconds of begin immersed in water. The swelling results in small pieces of the samples breaking off as swelling cracks propagate through the sample. The sample will eventually disintegrate completely and the more rapidly complete disintegration occurs, the greater the swelling and slaking potential.

Anhydrite recognition: Identification of gypsum is just as important as identifying anhydrite (refer to section 1.2 above). Both minerals can be scratched with a steel blade and gypsum can be scratched with a fingernail. To distinguish anhydrite from carbonates hydrochloric acid can be used to determine if effervescence occurs, which indicates carbonaceous material. It should be noted that in marine deposits a carbonaceous cement often occurs, so a pure white powder sample should be tested to minimize contamination from the surrounding rock mass.

If swelling rock is suspected core samples can be placed in a free swell test directly after being recovered to determine the magnitude of swelling strains. In section 2.4 below the different swell testing procedures are explained. Further core samples should also be preserved in an air
tight container or coated in paraffin wax for more detailed laboratory testing, as shales and mudstones have a tendency to break apart and deteriorate over time (Goodman 1993).

**A.1.5 Laboratory Analysis**

As part of the ground investigation program, core samples and bulk samples of the rock mass are collected and brought to the laboratory for further analysis. In the laboratory the mineralogical make up can be identified and the swelling potential examined in more detail.

In addition to the laboratory analysis outlined in more detail below, other rock mass properties are determined in the lab, such as unconfined compressive strength (UCS), rock mass modulus, poisons ratio, to name a few. These tests are not discussed in this report, although some correlation between mineralogical make up and rock mass strength can be determined.

**A.1.5.1 Mineralogical Identification**

Mineral identification can be done using several tools. Typically thin section analysis is conducted to determine sample layering and mineralogical arrangements; although with shale the fine grained nature makes it difficult to distinguish between grains. In this case, X-ray diffraction (XRD) and Scanning Electron Microscopy (SEM) can be done to determine mineralogical assemblages and relationships. XRD is a necessary tool to identify clay minerals present in the samples, as clay particles often lead to swelling. The type and amount of clay minerals can lead one to conclude that the swelling is being caused by the clay particles taking up water or if another swelling mechanism is taking place.
A.1.5.1.1 X-Ray Diffraction

The method of X-Ray diffraction (XRD) allows the user to determine what minerals are present in a sample of any grain size. Although coarse grained samples can be readily identified through other techniques, XRD is typically used on fine grained materials.

X-rays are electromagnetic radiation and are a shorter wave length then light. When the x-rays are directed at a particular mineral (incident ray), the x-rays are scattered. Most of the scattered x-rays are eliminated through destructive interference. At certain angles of incidence, constructive interference occurs for certain minerals and this diffracted pattern is unique for that particular crystal lattice (USGS 2001). Bragg determined that diffraction is related to the spacing of the crystal lattice and developed Bragg’s Law (USGS 2001) which states, $2d(sin\theta) = \lambda$, where $d$ is the lattice spacing, $\theta$ is the angle of incidence and $\lambda$ is the wave length of the characteristic wave. Figure A.1 below illustrates the meaning of Bragg’s Law. It is through the use of Bragg’s Law and the properties of constructive interference that XRD has become a tool for mineralogical study.

![X-Ray Diffraction Diagram](image)

**Figure A.1**: X-ray diffraction pattern illustrating Bragg’s Law.
A.1.5.1.1.1 Procedure

1. Sample is crushed to a -2 mm consistence with hammer or equivalent metallic crushing apparatus.

2. Crushed sample is placed in a micronizing mill with approximately 7 ml of ethyl alcohol.

3. The micronized sample is poured over a glass disk and the ethyl alcohol is allowed to evaporate off, leaving the powdered sample on the glass.

4. The glass disk is then placed in the XRD instrument and the sample is scanned from 2 to 70 (2θ) ° to get a whole rock spectrum.

5. The data is then analyzed to determine the appropriate minerals corresponding to the peak counts obtained by the XRD scan, with the aid of computer software which selects the most relevant minerals for the scan results.

A.1.5.1.1.2 Quasi-Quantitative Abundance Calculation

With the XRD a quasi-quantitative abundance calculation is possible using the area under the peaks to represent the relative abundance between the samples. This was done using only the primary peak, for each mineral in this study. A calculation example is presented in Appendix A.

A.1.5.1.1.3 An Example from the Queenston Formation

The XRD was used to detect whole rock chemistry in order determine the mineralogical make up. The results are primarily to be used for comparing mineralogy between previous swell tests and currently underway swell testing, which are the subject of a future report.

A comparison between all six samples tested show variations in minerals present from sample to sample. Sample S2 is missing the major calcite peak, suggesting that little to no calcite exists in this sample. Albite occurs in the Seal sample, where the others appear to have a very
weak peaks, suggesting only a trace amount. Halite only occurs in the fresh and preserved samples. Comparing abundance of each mineral between samples containing like mineralogy and considering the error associated with the calculation method (± 10%) there is only a weak difference between the percentages. The most notable being Muscovite, see Figure A.2 below.

The ‘other’ mineral in Table A.1 represents an unidentified peak, which occurs in all samples. The list of suggested minerals given by the computer analysis software was unable to identify this peak. Where listed as a possible mineral by the computer it has been noted in table 1.

![Figure A.2: Six sample XRD comparison. All samples have a similar mineralogical make up. All samples contain abundant quartz, clinochlore, and muscovite.](image)

<table>
<thead>
<tr>
<th>Peak List</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quartz: SiO2</td>
</tr>
<tr>
<td>Mica: K2[Al-Fe]+Si3O10(OH)2</td>
</tr>
<tr>
<td>Clinochlore: NaAlSi3O8</td>
</tr>
<tr>
<td>Muscovite: K[Al-Fe]Si3O10(OH)2</td>
</tr>
<tr>
<td>Albite, ordered: NaAlSi3O8</td>
</tr>
</tbody>
</table>

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Table A.1: XRD sample results showing the most abundant minerals, the percentage is based on the 100% peak for the listed minerals only. Note that montmorillonite and kaolinite peaks were not defined well and were there for not included in the whole rock percentages.

<table>
<thead>
<tr>
<th>Mineral</th>
<th>Old</th>
<th>Sealed</th>
<th>S2 swell</th>
<th>S3a fresh</th>
<th>S3b fresh</th>
<th>S3a swell</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quartz</td>
<td>32</td>
<td>45</td>
<td>31</td>
<td>35</td>
<td>47</td>
<td>27</td>
</tr>
<tr>
<td>Clinochlore</td>
<td>16</td>
<td>13</td>
<td>19</td>
<td>18</td>
<td>15</td>
<td>18</td>
</tr>
<tr>
<td>Muscovite</td>
<td>35</td>
<td>37</td>
<td>49</td>
<td>41</td>
<td>28</td>
<td>49</td>
</tr>
<tr>
<td>Calcite</td>
<td>5</td>
<td>3</td>
<td>-</td>
<td>1</td>
<td>4</td>
<td>3</td>
</tr>
<tr>
<td>Halite</td>
<td>10</td>
<td>-</td>
<td>-</td>
<td>3</td>
<td>3</td>
<td>-</td>
</tr>
<tr>
<td>Other</td>
<td>2</td>
<td>2</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>Montmorillonite</td>
<td>-</td>
<td>-</td>
<td>Trace</td>
<td>-</td>
<td>Trace</td>
<td>-</td>
</tr>
<tr>
<td>Illite</td>
<td>-</td>
<td>Trace</td>
<td>-</td>
<td>-</td>
<td>Trace</td>
<td>-</td>
</tr>
</tbody>
</table>

Clay minerals are better determined by a closer examination (by narrowing the 2θ range scanned) since they can be miss-identified for other clay minerals. A special treatment of the sample is necessary, such as glycolation and heat treatment to cause variations in the clay mineral crystal lattice thicknesses. The treatments have varying affects on different clay minerals and thus can help in identifying them more precisely. They still remain only a small fraction of the total mineralogical make up of the Queenston Formation.
A.1.5.1.1.4 Glycolation and Heat Treatment Procedure

The glycolation and heat treatment procedure causes changes in the clay mineral layer thicknesses and different clay minerals are affected differently by the process. By applying ethylene glycol to the sample and re-scanning the specimen a new peak count is obtained. By comparing the new peak count to the original (untreated) peak count for the same given $2\theta$ range the clay mineral can be identified by changes which occur. The process is further refined by cycling the specimen through heat treatment at 400 ºC and then 550 ºC, comparing the peak counts from each treatment cycle identifies the clay mineral based on its behaviour.

Figure A.3 below compares the different cycles of testing and shows only one peak, 14.5 degree peak, which changes after being heated to 550 ºC. All the other peaks remain constant throughout the testing which indicates that these peaks represent chlorite and illite. The remaining results for the clay identification are presented in Appendix A.A.
Figure A.3: Comparing clay identification cycles.

A.1.5.1.2 Scanning Electron Microscope

The scanning electron microscope allows the user to view their specimen at large magnifications. The instrument causes electrons to be accelerated between a cathode, often a tungsten filament, and anode plate with a hole in it. Some of the electrons accelerate past the anode plate, through the hole and on to the sample. The electrons hitting the sample deflect and produce secondary electrons and back scattered electrons, see Figure A.4.
Figure A.4: Incident beam – sample interaction and the resulting scatter types. Note that secondary electrons and primary backscattered electrons and x-rays are the typical emitted signals detected.

The secondary electrons are detected and converted to a voltage. As the machine scans, the changes in voltage cause a change in the intensity of emitted beam of light which is used to create an image on the computer screen. As the beam scans the sample, the intensity changes according to the topography of the sample and an image is produced based on the varying intensities.

Chemistry can be conducted on sample in a similar manner as in XRD by detecting x-rays which are diffracted through the sample.

The SEM is a useful tool for determining surficial differences between samples, utilizing the imaging feature. When a particular area of interest is observed, then chemistry can be conducted to determine the mineralogical characteristics at that location.
A.1.5.1.2.1 Procedure

1. Core sample is crushed with geological hammer to obtain rock fragments smaller than 10 mm.

2. The rock fragments are affixed to an SEM sample stand with a carbon based glue.

3. The sample is then coated with a conductive coating.

4. Samples are then placed in the SEM and imagining and chemistry can be conducted on whole fragment or individual grains.

A.1.5.1.2.2 An Example from the Queenston Formation

The primary interest in the SEM results is the imaging differences between different samples scanned. The images show the structure of the mineralogical arrangement and allowed for a comparison between fractured surfaces of swollen and non-swollen samples.

The fractured surfaces were created using a geological hammer and it should be noted that less force was needed to create small rock fragments out of the samples old and S3a swell. A low magnification comparison between all samples tested is presented in Figure A.5 at the end of this section. At low magnification the surfaces appear similar, although in Figure A.5B and Figure A.5C there are more areas of flat, smooth, non-stacked surfaces.

The smooth planer surfaces, shown in Figure A.6, are more common in Sealed and S3a fresh samples. Localized areas do exist in the Old and S3a swell samples, see Figure A.6. These surfaces appear to be more competent and allow the formation of a weak striation as seen in Figure A.6B. There appears to be more straight fracture planes within the fresh and sealed samples, see Figure A.6C. In samples which have experienced swelling, either from
submergence or atmospheric moisture, there appears to be sub-angular to sub-rounded corners at the junction of two fracture surfaces, see Figure A.7C.

In Figure A.6, all samples exhibit a stacked structure. In Figure A.7A and Figure A.7D, the stacked structure is throughout the whole sample and Figure A.7B and Figure A.7C it is only locally similar to the other samples.

There are differences between fractured surfaces examined in the SEM. Smooth and planar surfaces exist in the fresh samples, where as the swollen samples contain more platy surfaces.

Figure A.5: Low magnification surface images. A is sample Old, B is sample Sealed, C is sample S3a fresh and D is sample S3a swell.
Figure A.6: Comparing smooth, planer, competent surfaces. A is sample Old, B is sample Sealed, C is sample S3a fresh and D is sample S3a swell.
Figure A.7: Showing layered structure in all samples. A is sample Old, B is sample Sealed, C is sample S3a fresh and D is sample S3a swell. Note the rougher surfaces in B and C.

A.1.5.2 Swell Potential Testing

The swell potential testing should be conducted on preserved core or bulk samples which are suspected to have high clay content, gypsum-anhydrate or other swelling related minerals based on field observations and mineralogical studies.

There are many swelling tests which can be conducted. Figure A.8 below shows four such apparatuses which can be used to conduct swell testing. Traditionally oedometer tests, or confined test, Figure A.8b and Figure A.8c, have been used to determine the swelling potential
because the side wall confinement simulates the tunnel invert (Einstein 1996). Free swell testing apparatuses, (Figure A.8a), are an easy and cost effective way to determine the maximum swelling strain, (Figure A.9), although the effect of external pressures is not simulated. With increased confinement the time to reach the same strain is increased and in some instances the swelling in one principle direction can be reduced by confinement in the orthogonal direction (Hawlader et al. 2003).

![Figure A.8: Swell testing apparatuses. a. is a free swell test apparatus, b. is a semi-confined apparatus, c. is confined or oedometer apparatus and d. is a triaxial apparatus.](image)
Figure A.9: A comparison between oedometer and free swell testing results showing the strain versus time curves from ISRM 1989, published by Einstein (1996). Note the early suppression of the swelling strains in the oedometer test.

Triaxial testing has become the researching tool of choice for studying swelling rocks (Barla 1999) as the stress path through the entire project duration, from its natural state, to excavation, to support installation and long term behaviour can all be simulated.

The three testing apparatuses, the free swell, the oedometer and the triaxial cell, will be discussed in more detail below. These three are the most commonly used cells, although several researchers have refined or modified the standard cells to suit particular situations (Franklin 1984 and Lee & Lo 1989). The international society of rock mechanics suggests that the oedometer
and the triaxial test be conducted to determine swelling pressures and swelling stress strain relationships, respectively (Einstein 1994a).

The test durations vary according to the material being tested and the particular swelling mechanism taking place in the specimen. For calculation of the swelling potential according to Lee and Lo (1989) however at least 100 days are required, since the swelling potential is the slope of the curve between 10 and 100 days plotted on a semi-log graph. Each style of test should be run until the swelling strains or swelling pressures level off and are allowed to stabilize. This can be considerably different for various materials.

A.1.5.2.1 The free swell cell

The free swell testing apparatus is the simplest and easiest testing apparatus to determine the maximum swelling potential in the vertical axis. Sample preparation is minimal and testing can begin right at the drilling site for fast results since the cell is light weight and simple.

The free swell test is traditionally conducted on core samples, although hand samples can be used as well. The samples should be cut so that two parallel surfaces can be used for measurement of the axial strain. In the radial direction it is preferable to have a uniform surface so that there is uniform infiltration of the ambient fluid. The ambient fluid is necessary for the swelling to occur (Hawlader et al. 2003) and as such a similar fluid to the proposed site should be used so that more realistic swelling occurs during testing. Once the sample is cut and the test cell is filled with the appropriate ambient fluid, the sample can be placed in the cell. See figure 10 below for samples tested as part of this report by the author.

The specimen is placed in a container filled with water (the ambient fluid) and both ends of the sample are in contact with porous stones, which allow the water to be in contact with the
entire specimen, see figure 8a. The axial swelling strains are measured using a dial gauge, which is contact with the sample via a plunger. As the sample swells, the plunger is pushed upwards and the amount of upward movement is measured with the dial gauge. The sample is unconfined in the radial direction and is allowed to swell in all directions. The swelling strain is measured in the axial direction, as shown in Figure A.10, over time giving an idea of the reactivity of the sample.

Figure A.10: Free swell testing on Queenston Formation core drilled on January 19, 2007.
The Queenston formation was recently tested using a free swell cell. The results indicate that rapid expansion occurs within the first three days and following that the rate of swelling gradually decreases over time. Table A.2 below indicates that the swell potential for the samples tested ranges from 0.011 to 0.63 % strain / log cycle, although the total strain at the end of swell testing shows little variation which is to be expected as the samples come from the same approximate depth and location. The local variation in swell potential between 10 and 100 days can be explained due to crack propagation which begins around day 10 and reaches its maximum after 100 days according to Lee and Lo (1989), but is sample dependant which causes variation. The free swell test results are plotted in Figure A.11 below.

Two of the samples were left in longer and these samples began to show signs of increased swelling strain rate around 80 days. This is consistent with reports by Hawlader et al. (2003) who indicated that this is likely to occur due to swelling induced crack propagation through the samples, which increases the surface area for swelling to occur. Unfortunately the testing had to be stopped due to laboratory constraints.

Table A.2: Swell potentials for samples tested during January 30 to May 2, 2007.

<table>
<thead>
<tr>
<th>Sample</th>
<th>S1</th>
<th>S2</th>
<th>S3a</th>
<th>S3b</th>
</tr>
</thead>
<tbody>
<tr>
<td>Swell Potential (% strain / log cycle)</td>
<td>0.022</td>
<td>0.27</td>
<td>0.63</td>
<td>0.011</td>
</tr>
</tbody>
</table>
Figure A.11: Free swell test results for samples of the Queenston Formation. The samples were taken from a drill hole directly below the Whirlpool – Queenston disconformity.

A.1.5.2.2 The oedometer cell

The oedometer cell, often used for consolidation testing of soils (Franklin 1984) can be used to measure the swelling strain in the axial direction while applying a load in the same direction. If strain gauges are fixed to the confining ring and the confining rings are calibrated for various pressures, then a relationship between strain or stress on the confining ring and swell pressure can be determined.

The sample preparation for an oedometer test is more labor intensive as the specimen must be machined to fit a confining ring. The sample must be placed on a lathe and trimmed to the diameter of the confining ring. The machining of the sample has a threefold purpose, it first
removes the outer surface of the sample which has already begun to swell due to the drilling fluid and it also creates a smooth uniform surface which will be in contact with the confining ring. It also removes any preservation agents which were used to store the samples for a period of time prior to testing. After the samples are machined to the appropriate diameter, the ring is tacked in place and the ends are ground smooth and flush with the confining ring. The sample is then ready to be placed in the oedometer cell for testing.

The oedometer testing apparatus consists of a round cell which the sample sits in on top of a pore stone, see figure 8c. A second pore stone is place on top of the confining ring and sample. The porous stone allow water to filtrate through the sample and the confining ring restricts the infiltration to the axial end of the sample only. A cantilever system is used to load the sample. The loading of the sample is used to determine the swell potential as a function of applied stress. A dial gauge reads the axial deformation and strain gauges read changes in the confining ring, which correspond to pressures from the ring calibration.

The results indicate the amount of swell pressure which will act on linings or foundations and the stress limit at which point swelling is inhibited. Once the inhibiting stress is determined, then the corresponding limit of swelling is also known. This limit may change over the course of time if the swelling continues to damage the rock and extend the plastic zone.

A.1.5.2.3 The triaxial cell

The triaxial cell has been used to simulate three-dimensional loading of the stress path near a tunnel excavation on core samples to see how this affects swelling. In traditional free swell and oedometer tests lateral stress effects are neglected. It was only when lateral stresses were measured in oedometer and triaxial tests that shear failure simultaneously with swelling was
discovered (Einstein 1996). It is only with advances in data acquisition equipment that the triaxial cell has gained favour for swell testing, since traditional cells would require the direct placement of strain gauges on the sample, which is difficult for swelling material (Franklin 1984). In M. Barla’s Ph.D. thesis (1999) the triaxial cell used in his experiments was equipped with 8 different kinds of measurement tools.

Sample preparation is similar to that of the free swell testing apparatus. The sample must have two parallel ends, square to the long axis of the specimen and any preservation agent removed. The sample is then covered with a rubber membrane to protect the sample from the fluid used in the triaxial cell to load the sample in the radial direction.

Once the sample is prepared it is placed on the base pedestal of the triaxial cell, see Figure A.12 below, which has a hole in the middle for drainage of the ambient fluid. The top cap is place on the sample and the membrane is secured to create a seal between the sample and the end caps.
A data acquisition system is usually used to collect the data from a triaxial testing apparatus due to the precision of the results required and the number of items being monitored. The axial strain, radial strain, cell pressure and pore pressure at both ends of the sample can be measured using transducers which feed into a data acquisition system. A feedback loop can be used to monitor and maintain the cell pressure for example, so that the desired confining pressure is maintained. Such systems are complex and expensive, but yield valuable test results which can
be used throughout the pre face excavation, excavation, initial support, final lining and long term
design process.

**A.1.6 Design and Construction Considerations**

The main consideration when designing a structure on or in swelling rock is how will the swelling
behaviour affect the structure and how can the affects be minimized or eliminated. Foundations
and roads can heave or pop-up (Lindner 1976) and tunnel linings can crack due to the swelling
strains and pressures which act on the structure. These failures can cause expensive delays and
can create difficult repair conditions.

Hawlader et el (2003) states that for the Queenston Formation in Southern Ontario to
swell there must be the addition of fresh water and stress relief. These two criteria are common
for most swelling materials (Lindner 1976). As the stress relief will occur whether excavating a
few meters of overburden or excavating a tunnel at depth, the primary focus of the designer
should be the water-rock interaction.

**A.1.6.1 Surface Works**

For surface works the swelling phenomena can crack foundations and road surfaces and in sever
instances cause heave. Lindner (1976) points out that surface works can cause either an increase
in water content or a decrease in water content near the structure and in either case this
disturbance in equilibrium can cause swelling to occur.
Figure A.13: Idealized water content profile under a home after construction. Water is allowed to concentrate under the home because evaporation cannot take place. Take from Lindner (1976) after Jennings et al. (1958).

The presence of the structure at surface can prevent evaporation of surface water below the foundations causing an increase in the water content below the structure. In Figure A.13 the percentage increase in water content is shown by contour lines below the foundation of a structure.

The season of construction also has an influence on the rock mass for surface works. If constructing a foundation in a wet season the increased water content will cause swelling and
conversely creating a surface excavation in a dry season could cause the foundation rock to crack during drying, which will later increase the surface area for swelling to occur.

**A.1.6.2 Water Management at Surface**

To minimize swelling behaviour during and after construction the moisture content of the rock mass should be maintained at closely as possible to that of the in-situ conditions. This will minimize cracking due to drying and/or swelling during construction, which will also in the long term reduce the amount of fractures for ground water to move through and cause swelling after construction.

Adequate pumping facilities and drainage ditches should be used around the site so as to remove and reduce standing water. The capacity of the drainage system should also consider large rain events which could be possible throughout the construction phase of the project.

The heavy construction industry typically is designing structures to last 90 – 100 years and with life spans of this magnitude drawing down and maintaining a lower water table throughout the life a structure would be costly. Also since the structure is at surface, water infiltration is inevitable. For these reasons it is more desirable to incorporate void space, which can absorb the swelling pressures, into the design of the structure. Figure 14 below shows some methods of absorbing swelling pressure. The walls of the building foundation can be lined with compressible insulation and the foundation can be constructed with a false bottom so that swelling can occur into the void space. It has been found that swelling of the Queenston Formation can be eliminated by applying 4-5 MPa to core samples (Rigbey & Hughes 2007) during swell testing. In this case foundation piles could be used to concentrate the load of a large structure in such a manner as to suppress swelling as illustrated in Figure A.14 (Lindner 1976).
By eliminating swelling behaviour under the piles and leaving a void under the false foundation, swelling can occur locally between the piles and not damage the structure. The most cost effective means of dealing with swelling for surface works is to minimize swelling damage during construction and to absorb swelling pressures which are induced after the construction.

Figure A.14: Absorbing swelling pressures in foundation design, after Lindner 1976. The pile causes the load of the foundation to be concentrated at one location, this causes a stress build up above the critical swelling pressure, which inhibits swelling. Compressible material can be used under the foundation and along the side walls to prevent swelling damage.
A.1.6.3 Underground

Underground structures are more susceptible to swelling damage if not properly mitigated in the design process because they are fully enclosed in the swelling material. Steiner (1993) indicates that the most common damage in tunnels due to swelling is invert heave and crushing of strong inverts as occurred in the Jura mountain tunnels. In Southern Ontario two tunnels which have been monitored closely and results reported by Hawlader et al. (2005) indicate that tensile cracking of the final concrete liner occurred near the spring line. Damage to underground structures is more difficult to monitor and repair in the long term and it is there for necessary to eliminate or reduce the swelling impacts on the structure during construction.

During construction of underground works in swelling rock the collection and removal of water both ground water and production water should be done in a timely manner and so that it has little time to induce swelling in the freshly exposed rock surface.

The ground water chemistry plays an important role in the swelling mechanism. In Southern Ontario the ground water tested in the Queenston Formation is connate and supersaturated in chlorides and sulphates (Rigbey & Hughes 2007). As the Queenston Formation requires fresh water to swell, the in-situ ground water is at equilibrium with the rock mass, in terms of swelling. The high chloride and sulphate concentrations can lead to break down of traditional reinforced pre-cast concrete liners. For the current Niagara Tunnel Project a Flexible Polyolefine waterproof membrane (Rigbey & Hughes 2007) is to be placed prior to the final cast in place liner so as to eliminate the connate water interaction with the concrete and also to eliminate fresh water from leaving the tunnel and causing swelling. At other site the permeability of the over lying rock formation maybe higher allowing fresh surface water
to interact with the potentially swelling material. In this case installing a compressible material behind the final lining, which can absorb the swelling pressures, would be better. In pressure tunnels however a compressible material may not be appropriate if the pressure is able to crush the liner as the compressible material shrinks. Where a membrane or a compressible material is not feasible, a heavily reinforced concrete liner is recommended.

In road and rail tunnels, where the ground water drains towards the excavation, invert heave is the most likely difficulty. Seepage will be focused along the invert, either between the tunnel lining and the rock mass or through the damaged zone around the tunnel (Anagnostou 1995). This causes the invert to swell sooner than the rock mass in the walls or crown (Anagnostou 1995) resulting in invert heave as the primary failure mechanism. Strengthening the invert slab and giving a slightly curved shape which gives rise to a better stress distribution around the tunnel (Einstein 1993) helps to reduce the chance of invert heave. Kovari et al. (1988) show four methods of construction, (Fig 15), which can reduce or eliminate invert heave and the buildup of swelling pressure on the lining. Two of the support mechanisms are designed to be strong enough to resist swelling pressures and the other two allow some swelling strain to occur, which reduces the buildup of swelling pressure. Kovari et al. (1988) conclude that the implementation of yielding support elements is the most economical solution, see Figure A.15.
Figure A.15: Design measures used in swelling rock after Kovari (1988), tunnel sections with and without yielding supports. The invert arch and anchoring systems are designed to provide enough resistance against the swelling pressure. The open space and yielding support systems are used to absorb swelling strains.

When constructing underground excavations in swelling rock the ground water chemistry and the shape of the excavation are important considerations to the design. Specifically in rail tunnels where alignment constraints are very particular, invert reinforcement to provide adequate counter pressure to the swelling should be used. In instances of high swell potential where high swelling pressures can develop the only economic means of construction may be a yielding invert segment and a compressible liner.

A.1.7 Model Simulation

To gain a better understanding of how the swelling behavior will affect the structure in question numerical model simulations can be used. Extensive work has been conducted to determine an appropriate constitutive model for the swelling behaviour of shales from Southern Ontario. The
process has been summarized by Kramer and Moore (2005) and these authors propose a formulation of a finite element model, which has been implemented in the program AFENA.

The finite element model suggested by Kramer and Moore (2005) uses a “pseudo load vector”, $\Delta F^{*i}$, to calculate inelastic strains for the current time step. Figure A.16 below shows the approximation of the load vector for both the elastic and inelastic (nonlinear) portions. These vectors are used to determine the displacements, stress, strains and external load vectors for the model. The formulation is an approximation and does produce some drift between the calculated and actual values. This is corrected using an unbalanced force vector in the total load vector calculation.

Figure A.16: Direct Incremental Initial Strain Approach and use of a Pseudo-Load Vector
Figure A.17: Mutli-Kelvin element model used in the formulation of a finite element swelling model.

The equations used in the formulation presented by Kramer and Moore (2005) are based on three Kelvin elements links in series as seen in Figure A.17 above. The stress dependency of the swelling is modeled using a strain rate parameter which decays with time increments or stepping of the model. A single rate parameter is used to describe the strain decay for all three Kelvin units, since the rate parameter is neither stress nor direction dependant (Kramer & Moore 2005). The swelling parameters and the incremental stress vectors are rotated to the principle stress field where so that shear stress is not required in the calculation (Kramer & Moore 2005).

The above mentioned work by Kramer and Moore (2005) could be incorporated into modeling software such as FLAC, an Itasca program, through the use of FISH. Using FISH a “pseudo load vector” could be used to induce swelling if appropriate conditions were to occur in the rock mass at particular zones.

The use of numerical models to simulate swelling behaviour is still at the research level. Constitutive models suggested by Hefny, Lo, and Huang, (1996), Lo and Yuen (1981) and Lo and Hefny (1996 ) have been developed and refined to describe the swelling behaviour of the
Queenston Formation. These constitutive models could be used in design process, although with due caution to the assumption made by the authors.

**A.1.8 Concluding Remarks**

Swelling rocks present difficulties throughout the life of a project, right from the early stages of investigation and laboratory testing through to design and construction. During the investigation stage core samples must be preserved properly so that more accurate laboratory testing can be conducted. Prior to conducting strength and deformation laboratory testing, mineralogical identification should be conducted to determine if in fact swelling minerals are present and how these may vary across the site. The laboratory testing should include free swell and oedometer tests to determine the swell potential and the swelling pressures respectively. Also triaxial swell testing can be conducted so that excavation stress paths can be implemented to determine when swelling will occur and how destructive it can be. Moving into the design phase consideration for the ground water chemistry and flow paths can lead to the adoption of design methods which either eliminate and suppress swelling or absorb swelling pressures. Ground, surface and production water needs to be manage appropriately so that swelling during construction is minimized. This is especially important when tunnel linings have been designed to withstand swelling pressures as ground water will move along the damage zone typically located at the invert. If the damaged zone is deep, increased swelling pressure may occur and damage the invert slab. Throughout the life of the project considerations for the swelling mechanisms and how the swelling will interact with the structure should be considered. With new developments in numerical modeling, simulation of swelling behaviour will soon be a tool useful in predicting swelling behaviour during the life of the project. Swelling behaviour can be destructive during
construction and after many years after construction. There for it is necessary to use all tools available to gain a better understanding of the material before construction begins.

This report has been generated as a guide to those working on or in swelling rock. It is not a complete guide to excavation and tunneling, but is meant to help elaborate on the process of investigation through to design and construction on or in swelling rock.

References (Appendix A)


Lee, Y.N. and Lo, K.Y., 1989. The swelling mechanism of the Queenston shale. Geotechnical Research Centre Report, University of Western Ontario. GEOT-7-89: ISSN 0847-0626


Appendix A.A: X-Ray Diffraction Calculations
A.A.1 X-Ray Diffraction Abundance Sample Calculation

Calculation Steps:

1) Draw a tangent line along the bottom of the peak profile, without crossing any peaks. This will be the zero line.
2) Identify the 100 % peaks that represent the primary minerals.
3) Draw a triangle, which will represent the area under that peak, for all primary mineral peaks. The base of the triangle will be along the tangent line, or zero line.
4) Find the area under each primary peak.
5) Find the total area represented by the primary peaks by summing all the areas together.
6) Calculate the quasi-percentage each peak represents.
7) For muscovite there are two 100 % primary peaks. One of these peaks is part of the quartz peak. The area of the muscovite peak within the quartz peak must be subtracted in order to determine the area for quartz.

Figure A.A.1.1: Percent abundance sample calculations, showing the triangular area method of calculation.
Table A.A.1.1: Percentages of primary minerals in sample NF44

<table>
<thead>
<tr>
<th>Mineral</th>
<th>∆ Area</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quartz</td>
<td>2.133</td>
<td>31.555</td>
</tr>
<tr>
<td>Clinochlore</td>
<td>1.100</td>
<td>16.277</td>
</tr>
<tr>
<td>Muscovite</td>
<td>2.368</td>
<td>35.033</td>
</tr>
<tr>
<td>Halite</td>
<td>0.640</td>
<td>9.470</td>
</tr>
<tr>
<td>Calcite</td>
<td>0.360</td>
<td>5.327</td>
</tr>
<tr>
<td>Other</td>
<td>0.158</td>
<td>2.338</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>6.758</strong></td>
<td><strong>100</strong></td>
</tr>
</tbody>
</table>

A.A.2 Swell Potential Calculation

Free Swell Testing

Swell Potential = Slope = rise / run  
Swell Potential = \((1.42 - 0.4) / (100 - 3)\)  
Swell Potential = \((1.02) / (97)\)  
Swell Potential = 0.011

Figure A.A.2.1: Swell potential calculation for samples S3b.
Appendix B: Published Material
B.1: Engineering geology, glacial preconditioning and rock mass response to large scale underground excavation in the Niagara Region

Perras and Diederichs 2007
Engineering geology, glacial preconditioning and rock mass response to large-scale underground excavations in the Niagara Region

Matthew A. Perris, M.Sc.E. Candidate (Queen’s)
Hatch Energy, Niagara Falls, Ontario, Canada

Dr. Mark S. Diederichs, PhD., P.Eng.
GeoEngineering Centre at Queen’s – RMC, Kingston, Ontario, Canada

ABSTRACT: A new 10.3-km by 14.44-m diameter TBM-driven tunnel is under construction beneath the Canadian City of Niagara Falls, Ontario, descending through the entire stratigraphy of the Niagara Escarpment, including dolomites, limestones, sandstones and shales. The tunnel will run for much of its length through the Queenston Formation and will encounter mixed face conditions, blocky ground, horizontal and inclined shear surfaces, and high horizontal stresses, as well as unique challenges, such as swelling, vertically variable hydrological conditions and topographic effects beneath a buried gorge.

This large project provides the backdrop for a study of large-scale heterogeneity and anisotropy in geology, material properties and stress conditions, and the impact on excavation performance and rock-support interaction. An understanding of the digenesis, glacial preconditioning, erosion and structural development of the existing rock mass will help in reconstructing the complex stress regime, including vertically discontinuous lateral stresses and macro-structural components, laterally extensive shear lamellae and inclined shear structures near topographic extremes.

This paper will discuss the geological origins of these features through by numerically modelling past glaciations and subsequent erosion in highly heterogeneous laminated ground. It is the relationship between this complex stress history and the general excavation response which is of interest. Stepping through the geological history of the Niagara Region, from glacial loading in the Pleistocene to post-glacial erosion, using numerical models will allow for a better understanding of the mechanics involved in fracture formation under these loading and unloading conditions. The model results indicate that deformations can extend out from the gorge up to 1000 m in some of the geological formations.

1 INTRODUCTION

1.1 Engineering Geology and Forensic Modeling

A simplified version of a complex geological history, including multi-staged glacial loading and unloading, glacial and fluvial erosion, and glaciofluvial backfilling was modeled to determine how the stress field has developed, what effects the evolution has had on the sedimentary units, and the relative origin and lateral extent of the geological structures, which are found to exist. Understanding the evolution of stresses and calibrating to sparse measurements allows interpolation and extrapolation of pre-extraction stresses and prediction of distributed macrostructure within the near-gorge Niagara Region.

1.2 Geotechnical Engineering Projects

The City of Niagara Falls has generated hydropower for over a century. Power generation, on the Canadian side of the Niagara River, commenced in 1892 (OPG 2006). In 1925, the largest hydro power station (of that time), commonly known as the Sir Adam Beck 1, was brought into service and, in 1954, a second power station, Sir Adam Beck 2 (OPG 2006) and two tunnels were constructed. Recent upgrades at the Sir Adam Beck complex have increased the efficiency and allowed the new Niagara Tunnel Project to proceed into the construction phase (Fig. 1).

Numerous tunnels, for sewer systems and nuclear reactor cooling, exist in the rocks of the foreland of the Niagara Escarpment (Hawelker 2005). Future excavations at greater depths mandate an understanding of the geological past and stress development.

1.3 Niagara Tunnel Project

The new tunnel will divert an additional 500 m$^3$/s of water to the existing Sir Adam Beck generating facility (OPG 2006). The new tunnel will pass under the buried St. Davids Gorge, at roughly 150 m below surface, while previous tunnels had surfaced prior to this gorge. The tunnel will be constructed predominantly in the Queenston Formation. The Niagara Tunnel Project presents a unique opportunity to observe the Niagara sedimentary strata and allow for a more detailed understanding of
the geological behaviour and interaction of the heterogeneous rock units down and into the Queenston.

2 GEOLOGICAL HISTORY

2.1 Paleogeography and Stratigraphy

The Appalachian Basin and the Algonquin Arch influenced the deposition of the Ordovician and Silurian strata of the Niagara Region. The Appalachian Orogen provided clastic sediments and variations in sea levels controlled the depositional environment (Mazurek 2004); consequently, the strata in the Niagara Region vary between dolomites, limestones, sandstones, shales and interbedded zones of these rock types (Fig. 2). In the northwest a Precambrian basement high, the Algonquin Arch, remained a positive structure throughout Paleozoic time, which caused the sedimentary beds to onlap and overlaps this structure (Currie & Mackasey 1978). In the Niagara Region, the strata gently dips south at 6 m/km (Yuen 1992).

The Queenston Formation is Late Ordovician in age and is part of a compound deltaic, shallowing upward sequence (Stearn et al. 1979). The Formation is a red argillaceous mudstone with occasional siltstone interbeds (Rigby 1992) and, when exposed to fresh water, has significant swelling potential (Yuen et al. 1992).

Three Groups, the Cataract, the Clinton and the Albemarle overlay the Queenston Formation. Overlying the Clinton Group is the Decew, Lockport and Guelph Formations (Currie & Mackasey 1978).

The Cataract Group is Lower Silurian in age and deposited in deltaic and shallow marine environments (Currie & Mackasey 1978). It includes the Whirlpool, Power Glen and Grimsby Formations. The Whirlpool sandstone, which unconformably overlies the Queenston (Mazurek 2004), represents a transgressive state of a sea that washed and rounded quartz sand grains (Stearn 1979). With minor fluctuations in the sea level, the depositional material alternated between sand, clay and calcareous shell fragments forming the Power Glen and Grimsby Formations (Winder & Sanford 1972).

The Clinton Group is Middle Silurian in age and comprised of the Thorold, Neahga, Reynales, Irondequoit and Rochester Formations, which were deposited in a shelf edge environment (Winder & Sanford 1972). Reworked Grimsby detritus formed the Thorold Sandstone as sea levels increased. With continuing sea level changes in an intertidal and lagoonal environment, the Neahga & Reynales Formations were deposited (Winder & Sanford 1972). The Irondequoit Formation is a crinoidal dolostone and the Rochester Formation is a dolomitic shale (Winder & Sanford 1972).

Patch reefs, forming the lower Lockport Formation and regional reefs, forming the upper Lockport and Guelph Formations (Tesmer 1981), completely surrounded the Michigan Basin and allowed for the precipitation of evaporates of the Salina Formation (Sanford 1968).

The numerical modelling presented in this paper utilized the above formations. Below the Queenston Formation a conglomerate of rock mass properties denoted as the Georgian Bay Formation and the Precambrian were used in the modelling.

2.2 Continental Glaciation and Subsequent Erosion

There were four major continental glacial stages during the Pleistocene—the Nebraskan, the Kansan, the Illinoian and the Wisconsin, listed from oldest to youngest. The most recognizable stage is the Wisconsin, as till from this stage are relatively unweathered (Hough 1958). Estimates of Pre-Wisconsin ice thicknesses range between 2000 to 3000 m thick near Hudson Bay and during the Wisconsin Menzies (2001) reported thicknesses of 500 to 1300 m.

Erosional estimates reported by Mazurek (2004) after Cercone & Pollack (1991), indicate that multiple unconformities represent <100 m each. It
was assumed that 300 m of additional sedimentary rock existed above the current topography. This material was by the pre-Wisconsin glacial stages.

Spencer’s work from 1907 confirmed observations by Lyell (1845) that a buried gorge existed between the current Whirlpool and St. Davids village. Sediment descriptions by Abidi, et al (1992), confirm that glacial advances occurred many times during the filling of the gorge as marked by the interbedded glacial till. Constraining the erosion of the St. Davids Gorge is somewhat more difficult. Karrow & Terasmoe (1970) reported that erosion occurred during the Port Talbot interstidal of the Early Wisconsin, or the interglacial Sangamonian period, prior to the advance of the first Wisconsin ice.

Erosion by the Niagara River, starting at Lewiston, formed the Niagara River Gorge after the Wisconsin ice retreated (Pengelly 1996).

The depositional environment, orogenetic tectonic influences and most notably the glacial erosion and fluvial erosion have been key factors in shaping the geological environment of the Niagara Region. The numerical modelling presented in this paper considered the influence of these events, in an attempt to model the stress and structural development of the Niagara Region.

### Table 1. Rock Mass Properties used in the numerical models; the Salina to Precambrian properties were derived as an aggregate of the particular rock units and based on engineering judgement. The Guelph properties come from Mazurek (2004) and the remaining properties are derived from 1989 to 1992 investigations for the Niagara Tunnel Project. GSI (Marinos & Hoek 2000) is equal to RMR with orientation and water corrections removed.

<table>
<thead>
<tr>
<th>Formation</th>
<th>RMR / GSI</th>
<th>UCS (MPa)</th>
<th>$E_m$ (GPa)</th>
<th>$m_b$</th>
<th>$s$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Salina</td>
<td>n/a</td>
<td>100</td>
<td>3.09</td>
<td>0.100</td>
<td></td>
</tr>
<tr>
<td>Guelph</td>
<td>n/a</td>
<td>120</td>
<td>3.25</td>
<td>0.125</td>
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<tr>
<td>Lockport</td>
<td>70 / 80</td>
<td>151</td>
<td>3.68</td>
<td>0.135</td>
<td></td>
</tr>
<tr>
<td>Decew</td>
<td>69 / 79</td>
<td>128</td>
<td>3.31</td>
<td>0.097</td>
<td></td>
</tr>
<tr>
<td>Rochester</td>
<td>64 / 77</td>
<td>42</td>
<td>4.40</td>
<td>0.078</td>
<td></td>
</tr>
<tr>
<td>Irondequoit</td>
<td>72 / 82</td>
<td>106</td>
<td>3.68</td>
<td>0.135</td>
<td></td>
</tr>
<tr>
<td>Reynales</td>
<td>67 / 77</td>
<td>95</td>
<td>3.08</td>
<td>0.078</td>
<td></td>
</tr>
<tr>
<td>Neagha</td>
<td>56 / 66</td>
<td>14</td>
<td>2.97</td>
<td>0.023</td>
<td></td>
</tr>
<tr>
<td>Thorold</td>
<td>78 / 83</td>
<td>163</td>
<td>8.17</td>
<td>0.151</td>
<td></td>
</tr>
<tr>
<td>Grimsby</td>
<td>79 / 75</td>
<td>100</td>
<td>4.10</td>
<td>0.062</td>
<td></td>
</tr>
<tr>
<td>Power Glen</td>
<td>63 / 68</td>
<td>100</td>
<td>3.19</td>
<td>0.029</td>
<td></td>
</tr>
<tr>
<td>Whirlpool</td>
<td>85 / 87</td>
<td>216</td>
<td>9.43</td>
<td>0.236</td>
<td></td>
</tr>
<tr>
<td>Queenston</td>
<td>65</td>
<td>40</td>
<td>1.86</td>
<td>0.021</td>
<td></td>
</tr>
<tr>
<td>Georgian Bay</td>
<td>n/a</td>
<td>36</td>
<td>1.68</td>
<td>0.012</td>
<td></td>
</tr>
<tr>
<td>Precambrian</td>
<td>n/a</td>
<td>70</td>
<td>2.58</td>
<td>0.021</td>
<td></td>
</tr>
</tbody>
</table>

3 ENGINEERING CONSIDERATION

3.1 Rock Mass Properties

During the investigation and planning period for the Niagara Tunnel Project, much was learned about the in-situ stress field, swelling properties of the various units, squeezing ground conditions, rock mass strength and deformation properties. The unconfined compressive strength (UCS), the rock mass Young’s Modulus ($E_m$) and the Hoek–Brown (1998) failure parameters, $m_b$ and $s$, used in the numerical modeling have been summarized in Table 1. The Grimsby and Power Glen Formations are interbedded sequences.

3.2 High Horizontal Stresses

Throughout Southern Ontario, high in situ horizontal stresses exist in the sedimentary rocks, locked in due to tectonic activity during the Appalachian mountain building, from sedimentary basin effects and from glacial loading and erosion.

Horizontal to vertical stress ratios ($K_o$) in the order of 3 to 5 have been measured in the Niagara Region for the Queenston Formation, with greater ratios existing in the formations above (OPG 2005). There can also be sharp differences in the stress state from formation to formation due to differences in elastic properties (Haimson 1983). The principle stress in the Queenston Formation, trending ENE near the buried St. Davids Gorge, are in the order of 9 to 23 MPa (Yuen 1992).

Table 1. Rock mass properties used in the numerical models; the Salina to Precambrian properties were derived as an aggregate of the particular rock units and based on engineering judgement. The Guelph properties come from Mazurek (2004) and the remaining properties are derived from 1989 to 1992 investigations for the Niagara Tunnel Project. GSI (Marinos & Hoek 2000) is equal to RMR with orientation and water corrections removed.

Figure 2. Geological stratigraphy, modified after Haimson 1983.
3.3 Structural Features

Three major physiographic features, the Niagara Escarpment, the Niagara River Gorge and the buried St Davids Gorge, control the structural style within the immediate vicinity.

Bedding plane fractures are predominant in the Niagara Region and vertical and horizontal jointing increases near the gorge due to tensile stress relief (Novakowski and Lapcevic 1988).

In the Queenston Formation, slickensided shears are a ubiquitous feature (Russell & Harman 1985). Horizontal shears also exist and are sometimes filled with silt (Rigby et al. 1992). The silt-filled discontinuities are close to the Whirlpool contact and decrease with depth (Rigby et al. 1992).


3.4 Excavation in Heterogeneous Sedimentary Strata

The geological setting and structural features of the Niagara Region pose a challenge for the construction of underground openings, although early pilot excavations have been undertaken in support of the Niagara Tunnel Project (Fig. 3). The stability of the excavation must be given careful consideration in both the short and long term.

Short-term considerations related to the immediate stability of the rock mass include the orientation of bedding planes and other discontinuities, the infilling material and groundwater. While the more massive formations pose fewer challenges (Fig. 3, bottom). In the latter case, vertical or inclined discontinuities intersect bedding planes to create blocks of intact rock. These blocks, when exposed in the crown of an underground excavation, require immediate support as apparently large elastic deflections in the crown serve to delaminate and fracture the bedding planes. With closely spaced bedding planes and vertical jointing, thin slabs can form in the crown. In high horizontal stress conditions, the closely spaced bedding planes create “beams” and, if the bedding is horizontal to subhorizontal, these beams can buckle under the stress concentration and self-weight. The Heart Lake sewer tunnel, near Toronto, Ontario, encountered excessive overbreak in a highly fissile rock (Lo 1979). Such overbreak can cause delays in construction and incur extra costs for the project when not adequately controlled by ground support.

Long-term or time-dependant considerations include creep of the rock mass and swelling of argillaceous rock units (Pan & Dong 1991). Creep of the rock mass is dependant on its rheological properties, and the magnitude and orientation of the stress field. Argillaceous rocks, as well as rocks containing anhydrite, are susceptible to swelling. The process is a result of oxidation of pyrite or the hydration of clay and/or anhydrite particles (Lieszkowszky et al. 1994) resulting in a volume increase. Duncan et al. (1968), determined that not all shales have a high swell potential and those with a lower swell potential are usually cemented with calcareous material.

These processes control deformation and affect the initial and final support, and careful timing of their installation is required for effective results.

4 NUMERICAL MODELING

In this paper, numerical modelling is used as a tool to look at how the past has affected the current rock conditions in the Niagara Region. The stress history and structural deformation in the Niagara Region due to tectonic, glacial and erosional events, has been simulated to determine possible origin and
lateral extent of structural features, which need careful consideration in geotechnical design.

4.1 Model Stages and Assumptions

The complex history of the Niagara Region, including continental glaciations and gorge erosion, was modelled using 35 stages in the software package Phase2, a RocScience program. As full non-linear 3D modelling could not practically provide the constitutive or geometric detail required for this study, four different sections have been modelled, two along the Niagara River Gorge and two along the buried St. Davids Gorge, to determine the affect gorge depth has on lateral extent of deformation.

The modelling starts with Pleistocene glaciatio, grouping the Nebraskan, the Kansan and the Illinoian, as one glacial advance and retreat for the purposes of the modelling, which utilized a distributed load to simulate the weight of the ice sheet as it advanced. A slight angle of 5° from vertical was used to represent the traction of the ice load on the ground surface as it advanced.

The ice thickness, load distribution, increased in four equal steps for a maximum ice load of 2000 m and sequentially advanced across the modelled section in four stages. At the peak of the ice thickness, the Salina Formation was excavated and the load immediately re-established on the surface of the Guelph Formation and glacial retreat followed.

In the Sangamon interglacial period, the St. Davids Gorge was eroded in five stages to imitate progressive river erosion. Sediment infilling was completed in one stage as the sediment had little affect on the surround rock mass. In the Niagara River Gorge sections, the advance of Wisconsin ice followed the earlier Pleisocenc glacial retreat.

The Wisconsin ice sheet, thinner than the earlier glacial stages, was simulated in the same manner, with four equal increases in thickness of 250 m and advancing across the section in four stages. Again, at the peak of the ice thickness across the model section, the rock surface was eroded to the present day topography, including the thickness of overburden sediments as rock. The overburden was assumed to be relatively thin in the Niagara Region and as such have little effect on the model results. Following the Wisconsin retreat, the Niagara River Gorge was eroded progressively in five steps to simulate river erosion.

The present day rock mass properties have been used in the numerical modelling (Table 1) and, with the exception of the Neagha Formation, all formations are stable at the start of modelling. The Neagha Formation could potentially have been slightly stronger in the past, but through glacial loading and unloading, erosion and stress development, the strength may have been reduced.

The current modelling ignored pore pressure. Future models should incorporate pore pressure as it will have an affect on extending the deformation zones beyond the current dry conditions modelled, especially during glacial loading. The results of this model should be viewed as a lower bound for overall deformations and lateral disturbance extend away from the gorges. Pore pressures will enhance and extend the plastic deformations and the interbed localization of shear illustrated in these scoping models.

The assumptions in modelling the geological history of the Niagara Region do have influences on the model results. Future work should include a sensitivity analysis on the various inputs, such as glacial loading increments, pore pressure, lateral thrust tractions due to advancing ice and erosion style.

4.2 Model Stress Field and Deformation Results

The stress field was initially set at a maximum Ko ratio 3.5 perpendicular to the Appalachian front and 2.5 parallel, with locked in stresses of 2 MPa in both directions. Target model stresses, after erosion, of 17 to 24 MPa were used at elevation 0 (masl) to determine the above mentioned initial stresses. In the 2D models, erosion effects of one gorge on the other gorge have not been represented. Several combinations of stress ratio and locked stresses are possible and the model results are correspondingly non-unique, although they are a best fit at elevation 0.

4.3 Vertical Stresses

Haimson (1984) reported that the vertical stress calculated from hydrofracturing was 0.7 MPa greater than the assumed vertical stress based on the weight of the overlying rock. The model results show that there is a zone between 100 and 400 m from the edge of the gorge in which the vertical stress is greater than the weight of the rock mass, in the order of 0.25 to 0.5 MPa (Fig. 4). Beyond 400 m from the gorge edge, the model vertical stress and unit weight calculation are similar.

4.4 Horizontal Stresses and Tensile Failure

High horizontal stresses are a characteristic of the Niagara Region. This stress field locally adjusts to topographic features, such as the Niagara River Gorge, causing stress concentrations at the base and relaxation on the sides. The model results indicate that the horizontal stress field begins to adjust on
average 350 m (at surface) from the edge of the gorge.

As expected, a relaxed (tensile yield in the model) zone around the gorges resulted during the erosion of the gorge, as indicated in the model results in Figure 5. In this zone, shear failure has also occurred.

The relaxed/tensile zone around the gorges is most laterally extensive in the Neagha Formation, extending out from the gorge face 350 m (Fig. 5). Excluding the Neagha Formation, the tensile zone is more extensive in the formations directly above a shale layer, which can be expected based on the contrasting rock mass stiffness. Modelled tensile failure in the Queenston is isolated to within 50 m of the gorge face, for all sections except the deep section at St. Davids Gorge where the tensile zone extends 100 m away, suggesting that there is a relationship between the depth of gorge erosion and the lateral extent of tensile failure structures. However, the elements, which failed in tension, do not extend far vertically below the gorge bottom. Further investigation of the effect of glacial pore pressures and lateral glacial thrusting, which could affect the depth of damage, should be considered.

The tensile zone is somewhat related to the geometry of the gorge itself. When the gorge base exists in a more competent rock formation, the tensile failure zones is less laterally extensive than when the flat is located in a less competent rock formation, as seen when comparing Sections B and C (Fig. 5). The tensile zone around the edge of the gorge creates loose material and block movement above the affected zones, which contributes to erosion and can be difficult when constructing near the edge of the gorge.

4.5 Maximum Shear Strain

The maximum shear strain contours in Figure 5 indicate that there is more lateral movement below the gorges, than vertical movement. There is some variation in the contour pattern between the model sections, which is most likely controlled by the geometry and depth of gorge erosion.

Cross Section A (Fig. 5), representing a narrow gorge, shows a very small zone of shear strain, suggesting that the width of gorge erosion controls
the lateral extent of deformation. In Sections B, C and D (Fig. 5), the zone of shear strain is much different. Section B (Fig. 5), the zone of shear strain suggests that existing shear planes would be flat lying and slightly inclined close to the ground surface. In Figure 5, Sections C and D, the shallow and wide gorges form a zone of subhorizontal bulbous shear strain, which is inclined toward the gorge. This suggests that any shear planes that exist are inclined at a subhorizontal angle up toward the gorge. The lateral extent of the zone in all model sections (except Section B), is contained within the width of the gorge at surface.

In Section B (Fig. 5), the zone of shear strain is wider than the gorge, suggesting that, with increased depth of erosion, the zone of shear strain is larger. Also, considering the other sections, the zone of shear strain becomes less inclined with increased erosion into the Queenston.

Development of shear planes relieves the shear strain. In the exploratory drilling and excavation for the Niagara Tunnel Project, horizontal shear planes existed in the Queenston away from the gorge (Fig. 3) and inclined shear planes existed underneath the buried St. Davids Gorge. The model results suggest the lateral extent and the inclination of shear planes in the Niagara Region are closely related to the depth and width of gorge erosion into the Queenston Formation.

5 DISCUSSION

5.1 Practicalities of Model Results

The tensile failure zone and the shear strain zone are typical features associated with gorge erosion in high horizontally stressed environments. The tensile failure of the more competent rock mass units directly above the shale units is a common occurrence and results due to the stiffness contrast. If the two units are cemented together, the stiffer unit will not be able to deforming to the same extent as the softer unit without failing. Essentially, the softer unit rips the stiff unit apart, causing tensile failure.

The deeper the erosion into the Queenston Formation, the more laterally extensive the tensile failure zone is in the Whirlpool Formation. The St. Davids deep section, 100 m of Queenston eroded, indicates that the tensile zone extends up to 1000 m from the gorge face, whereas in the shallower section, the distance is 150 to 200 m. This is a substantial variation suggesting a relationship between the deep and shallow sections in terms of the lateral extent parallel to the gorge.

Novakowski & Lapevic (1988) indicate a weathered-fractured network on the bedrock surface in the Niagara Region. There is only limited surface tensile or shear failure in the model results at the surface due to the smooth nature of the topography used in the model. This is assumed to have little effect on the deeper failure mechanisms in the Queenston Formation. If a undulating rock surface was modelled, increased surface deformation would develop.

Russell & Harman (1985) indicate that, within the Queenston Formation, curved microfractures and slickensided curved shears exist as a ubiquitous feature. The model results indicate that extensive shear failure has occurred in the Queenston Formation following glacial unloading. Another observation made in a test adit for the new Niagara Tunnel Project was laterally extensive shear planes. The shear strain indicates that sub horizontal-to-horizontal shear planes may develop during gorge erosion.

Modelling of the swelling phenomenon has not been included in this paper. Within the Queenston Formation with the addition of fresh water, the swelling can adversely affect engineering projects. The swelling zone would be controlled by the amount of fracturing due to excavation and the joint spacing.

The current decoupled models indicate that there is relationship between depth of erosion and the lateral and vertical extent of deformation. Future work with 3D modelling would result in a better understanding of the lateral extent parallel to the gorges of such features as the depth of gorge erosion changes.

5.2 Implications for Underground Excavations

The model results indicate that extensive failure near the Niagara River Gorge and the buried St. Davids Gorge exists. During the construction of large-scale excavations near these topographic features, blocky ground must be considered, as indicated by the tensile zone, even in the most competent units like the Whirlpool sandstone. The tensile failure indicated in the model would most likely result in vertical jointing in nature. These vertical joints could develop wedges in the crown of large-scale excavations.

A relationship exists between the lateral extent of the tensile zone and the depth of gorge erosion. The deeper gorge sections have a more laterally extensive tensile zone. When determining the location of a large-scale excavation, the depth of gorge erosion in the immediate vicinity deserves careful consideration.

The limestone and sandstone formations of the Niagara Region are the most competent units for excavation. The failure zone around the gorges is limited to less then 1000 m in all formations and on average is 350 m. Such formations would make
excellent material to have in the roof of an underground excavation.

The main concern is the placement and orientation of an underground structure in relation to the gorges of the Niagara Region. Careful consideration should be taken with respect to the failure zone around the gorge and adequate measures applied to support the rock mass for long-term stability in a high horizontally stressed environment.

6 CONCLUDING REMARKS

By modelling the complex geological history of the Niagara Region, the possibility of laterally extensive failure zones related to gorge erosion has been determined. Considering the high horizontal stresses, the geological heterogeneity and the contrasting rock mass properties, glacial loading and unloading, and erosion by glacial and fluvial action, the model demonstrates that the failure zone is on average 350 m from the gorge face and can be up to 1000 m. A relationship between the depth of gorge erosion and the lateral extent of the failure zone was evident. Future work should be conducted to construct a 3D geological model of the Niagara River Gorge, particularly the block bounded by the Niagara River Gorge, the buried St. Davids Gorge and the Niagara Escarpment, and to incorporate glacial pore pressures and lateral glacial thrusting.

7 ACKNOWLEDGEMENTS

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B.2: Modelling circular excavations in horizontally laminated ground: The role of rock and lamination strength properties

Perras and Diederichs 2009b
Tunnelling in Horizontally Laminated Ground:
Intact Rock and Lamination Strength
Perras, M.A. & Diederichs, M.S.

Department of Geological Sciences and Geological Engineering,
Queen’s University, Kingston, Ontario, Canada

"M.Sc. Eng. Candidate
"Ph.D., P.Eng.

The Niagara Tunnel Project
The Niagara Tunnel Project is a 14.4km diameter water diversion tunnel being excavated by a Tunnel Boring Machine (TBM) under Niagara Falls, Ontario (left). The rocks of the Niagara Region are part of the Appalachian Sedimentary Basin and divided into four groups, predominantly made up of limestone, dolomites, sandstones and shales, with unconfined compressive strength (UCS) from 14m to 214MPa. The depth of the tunnel, which is driven by St. David’s Buried Gorge, which was filled during the last glacial period (Perras & Diederichs, 2007). When completed, the tunnel will pass 500m3 of water to the existing St. Adam Rock Generating Station.

(B) Multi-beam Coupling
In this zone multiple beams interact together to create a composite laminated beam. Stresses flow through the composite beam and plastic yielding is not limited by individual laminations. As the lamination thickness decreases in this zone, the degree of tensile yielding in the launch area increases and the height of shear yielding increases above the crown.

(C) Stress Channeling
The crown deflections deviate from the isotropic behaviour because the stresses flowing around the excavation are channelled through the first beam above the excavation. This increases the amount of plastic yielding within the first beam which reaches a maximum along the line between areas (C) and (B) (center). The drop in crown deflections at this transition is a result of partial plastic yield of the second beam above the excavation. The deflections do not resume increasing until full multiple beam yielding occurs.

(D) Isotropic
A laminated rock mass has strength properties which are directionally dependent and this is termed an anisotropic rock mass. As the lamination thickness increases, the anisotropic influence becomes less. In this region the anisotropic crown deflections are similar to the equivalent isotropic (no laminations) crown deflections (lower). To compare model the isotropic rock modulus is reduced using 

\[ E_{iso} = \frac{E_{aniso}}{1 - v_{aniso}^2} \]

where \( E \) is lamination thickness and \( K \) is lamination normal stiffness (Brady & Brown, 2009).

References & Acknowledgements

Rock and Lamination Properties

<table>
<thead>
<tr>
<th>Geomechanic Properties</th>
<th>Rock Properties</th>
<th>Lamination Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lamination Thickness</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rock Properties</td>
<td>Beam Modulus</td>
<td>Beam Modulus</td>
</tr>
<tr>
<td></td>
<td>(GPa)</td>
<td>(GPa)</td>
</tr>
<tr>
<td></td>
<td>UCS (MPa)</td>
<td>Tension (MPa)</td>
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<tr>
<td></td>
<td>Friction Angle</td>
<td>Normal Stiffness (GPa)</td>
</tr>
<tr>
<td>60</td>
<td>70.0</td>
<td>6.40</td>
</tr>
<tr>
<td>120</td>
<td>7.33</td>
<td>100.0</td>
</tr>
<tr>
<td>180</td>
<td>100.0</td>
<td>11.0</td>
</tr>
<tr>
<td>210</td>
<td>150.0</td>
<td>11.0</td>
</tr>
</tbody>
</table>

Summary of Findings

Four zones of crown deflection behaviour exist:
A. Plastic Yield Stabilization
B. Multi-beam coupling
C. Stress Channeling
D. Isotropy

- As the intact rock mass and joint strength properties increase, the lamination influence decreases
- Deviation from isotropic behaviour is independent of strength properties

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B.3: Tunnelling in Horizontally Laminated Ground

Perras and Diederichs 2009a
ABSTRACT: With growing populations, infrastructure projects are utilizing underground space to expand transportation networks, water supply and sewer disposal systems and hydropower facilities. These new projects require large diameter tunnels to meet the growing demands. As the tunnel diameter increases, not all rock mass properties are scalable and precedent experience becomes less reliable. One such non-scaleable property is lamination thickness. Numerical modelling was conducted to determine the influence of lamination thicknesses on a 16 m diameter tunnel. Five different areas of excavation response were found to exist for laminations ranging between 0.16 to 16 metres in thickness. For this tunnel diameter, critical lamination thicknesses of 6 metre and 1 metre were found to exist. Above 6 metres the excavation response was similar to isotropic models. Below 6 metres the stresses and yield are channeled by the lamination boundaries, increasing plastic deformation. Below 1m multiple lamination units become unstable and the displacements increase significantly and the shape and extent of the yield zone size changes dramatically. The yield zone is controlled by bed deflections and bed parallel shear. As the lamination thickness decreases tensile failure first begins in the haunch area and progressively extends above the crown until a self-limiting plastic yield zone shape is reach at lamination thicknesses below 0.4 metres. Conclusions drawn from observations at the Niagara Tunnel Project are those of the authors.

1 INTRODUCTION
Parallel laminations within a rock mass create anisotropic ground conditions, whether the laminations are caused by sedimentary bedding, joints or a tectonic fabric. Anisotropy is both modulus and stress dependent (Chappell 1990) and therefore not all rock masses with parallel structure or fabric will exhibit the same degree of anisotropic behaviour. Laminations can occur in all rock types and although most often thought of as a sedimentary feature, analogous flow behaviour can occur in igneous and metamorphic rocks. This flow behaviour can create parallel fabrics. Parallel jointing, caused by cooling in igneous and metamorphic rock or tectonic forces, can create anisotropy in a rock mass as well.

There are many methods available to the engineer for excavation and support design. The rock mass is classified, commonly by Q, RMR, or GSI (Barton 1974, Beiniakski 1974 or Hoek & Brown 1997, respectively) and intact rock properties are determined by laboratory testing. Using empirical relationships the intact rock properties are related to rock mass properties accounting for discontinuities and weaknesses in the rock mass via the classification scheme. One such relationship for modulus is presented by Hoek and Diederichs (2006) to determine a deformation modulus, $E_{rm}$, for the rock mass using GSI, USC, $E_i$ and $m_i$. This method has limitation when applied to anisotropic conditions.

Marinos and Hoek (2001) modified the GSI chart for flysch type rock masses, which essentially reduces the GSI value of the laminated anisotropic rock mass, from its GSI value on the original chart. This accounts for the reduced rock mass strength due to the laminated nature of
the rock mass, but does not account for excavation behaviour directly related to lamination parallel displacements and deflections.

Anisotropy is stress dependent (Chappell 1990) and as the stress in the rock mass increases the voids along laminations close, making the rock mass behaviour more isotropic by increasing the resistance to slip. Anisotropic behaviour is also affected by the intact rock modulus for the rock between the laminations. The laminations absorb much of the strain energy during excavation and intact rock bridges control the resistance to slip along the lamination. If the rock bridges are soft, more strain can occur than if the rock bridges are stiff. The rock mass property limits for anisotropic behaviour are not well defined in the literature and this paper presents steps to defining practical limitations for excavation design purposes.

2 LARGE SCALE UNDERGROUND EXCAVATIONS

Sedimentary rocks make up a large portion of the near surface formations around the world and typically these rocks exhibit a laminated structure as either bedding features or bedding parallel jointing. When the structure lies near horizontal the optimum excavation crown shape is flat and this is easily achieved with drill and blast methods. However, with the length and size of excavations being proposed and constructed today, TBM excavation is the preferred method. One such large scale project in horizontally laminated ground is the current Niagara Tunnel Project in Niagara Falls, Ontario, Canada.

2.1 The Niagara Tunnel Project

In the Canadian city of Niagara Falls, power generation commenced in 1892 (OPG 2006). In 1922, the largest hydro power station of that time, commonly know as the Sir Adam Beck 1, was brought into service and, in 1954, a second power station, Sir Adam Beck 2 (OPG 2006), including its two large water supply tunnels was placed in service. Recent upgrades at the Sir Adam Beck complex have increased the efficiency and discharge capacity, and allowed the new Niagara Tunnel Project to proceed into the construction phase, which when completed will divert an additional 500 m$^3$/s of water to the existing Sir Adam Beck generating facility (OPG 2006).

The new tunnel has passed under the St. David's Buried Gorge, at roughly 140 m below ground surface, crossing down through the Lockport to Whirlpool Formations and into the upper 60m of the Queenston Formation and is now making its ascent back toward the surface (Fig. 1). The Niagara Tunnel Project presents a unique opportunity to observe the Niagara sedimentary strata and allow for a more detailed understanding of the geological behaviour and interaction of the heterogeneous rock units above and into the Queenston Formation.

2.1.1 Geology Overview

The Appalachian Basin and Arch influenced the deposition of the Ordovician and Silurian strata of the Niagara Region. The Appalachian Orogen provided clastic sediments and variations in sea levels controlled the depositional environment (Mazurek 2004); consequently, the strata in the Niagara Region vary between dolomites, limestones, sandstones, shales and interbedded zones of these rock types. The sedimentary beds dip gently south at 6 m/km (Yuen et al 1992).

The Queenston Formation is Late Ordovician in age and is part of a compound deltaic, shallowing upward sequence (Stearn et al. 1979). The Formation is a red argillaceous mudstone with occasional siltstone interbeds (Rigbey et al 1992) and, when exposed to fresh water, has significant swelling potential (Yuen et al. 1992 & Rigbey & Hughes 2007). Three Groups, the Cataract, the Clinton and the Albemarle overlay the Queenston Formation.

The Cataract Group is Lower Silurian in age and deposited in deltaic and shallow marine environments (Currice & Mackasey 1978). It includes the Whirlpool, Power Glen and Grimsby Formations. The Whirlpool sandstone, which unconformably overlies the Queenston (Mazurek 2004), represents a transgressional state of a sea that washed and rounded quartz sand grains. With minor fluctuations in the sea level, the depositional material alternated between sand, clay and calcareous shell fragments forming the Power Glen and Grimsby Formations (Winder & Sanford 1972).
The Middle Silurian Clinton Group is comprised of the Thorold, Neahga, Reynales, Irondequoit and Rochester Formations, deposited in a shelf edge environment (Winder & Sanford 1972). Reworked Grimsby detritus formed the Thorold Sandstone. With continuing sea level changes in an intertidal and lagoonal environment, the Neahga & Reynales Formations were deposited (Winder & Sanford 1972). The Irondequoit Formation is a crinoidal dolostone and the Rochester Formation is a dolomitic shale (Winder & Sanford 1972).

Patch reefs, forming the lower Lockport Formation and regional reefs, forming the upper Lockport and Guelph Formations, completely surrounded the Michigan Basin and allowed for the precipitation of evaporates of the Salina Formation (Sanford 1968).

<table>
<thead>
<tr>
<th>Age Group</th>
<th>Formation</th>
<th>Brief Description</th>
<th>Approx. Thickness (m)</th>
<th>Symbol</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ordovician</td>
<td>Cataract</td>
<td>Albemarle Clinton</td>
<td>Grey crystalline dolomitic limestone</td>
<td>16.8 - 20.3</td>
</tr>
<tr>
<td>Lower Silurian</td>
<td>Grimsby</td>
<td>Thorold</td>
<td>Grey to reddish dolomitic limestone</td>
<td>1.2 - 3.1</td>
</tr>
<tr>
<td>Middle Silurian</td>
<td>Neahga</td>
<td>Green shale</td>
<td>Dark grey calcareous dolomite interbedded</td>
<td>2.4</td>
</tr>
<tr>
<td></td>
<td>Reynales</td>
<td>Rochester</td>
<td>White sandstone</td>
<td>3.6</td>
</tr>
<tr>
<td></td>
<td>Irondequoit</td>
<td>Light grey crystalline dolomitic limestone</td>
<td>11.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Decew</td>
<td>Light grey crossbedded sandstone</td>
<td>3.6 - 7.6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Power Glen</td>
<td>Grey shale to white calcareous sandstone</td>
<td>11.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Whirlpool</td>
<td>Green, irregularly bedded sandstone with red shale interbeds</td>
<td>16</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Queenston</td>
<td>Light grey crossbedded sandstone with red shale interbeds</td>
<td>5.6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Queenston</td>
<td>Red shale and argillaceous limestone</td>
<td>3.6 - 7.6</td>
<td></td>
</tr>
</tbody>
</table>

Figure 1: Physiographic features of the Niagara Region (modified from Novakowski and Lapcevic 1988), with inset map showing regional setting (Mazurek, 2004), and photo of TBM ready for launch (courtesy of OPG). Geological stratigraphy on right colour coded to match Niagara Tunnel Cross Section below.
2.1.2 Excavation Stability

Currently the Niagara Tunnel Project is making its ascent back into the Silurian strata after passing under St. David’s Buried Gorge in fall 2008. Overbreak was encountered in the Rochester, Neagha, Grimsby, Power Glen and Queenston Formations. The various rock mass properties were presented by Perras and Diederichs (2007) and Rigbey and Hughes (2007) and the reader is referred to these papers for further descriptions of the geology of the Niagara Region. As well, Mazurek (2004) gives a good overview of the rocks of Southern Ontario in a report for nuclear waste storage. Typical overbreak profiles to date are shown in Figure 2.

The stress ratio, $K_o$, in the Queenston ranges from 3 to 6, with the principle stress ranging from 9 to 23 MPa (Yuen 1992). Major physiographic features in the area, such as the Niagara Gorge, St. David’s Buried Gorge and the Niagara Escarpment, locally influence the stress field. The portal to the Niagara Tunnel is located within a block of rock bounded by the physiographic features mentioned above, which has reduced the stress from the regional levels. Most of the jointing in the project area is also associated with the physiographic features, such as observed vertical jointing and shear features under St. David’s Buried Gorge. With the high horizontal stresses the instability is focused in the crown, haunch and invert. Sidewall overbreak was limited in the upper units above the Queenston Formation and associated with random vertical jointing. Overbreak in the Queenston Formation can be broken down into 4 zones, as outlined in the tunnel longitudinal section of Figure 1.

The Rochester Formation is a dark grey calcareous shale – dolomite interbed, which is roughly 19 m thick at the project site. Bedding is difficult to observe in hand sample, with the exception of occasional slight colour changes. The formation is a stiff, yet lower strength rock mass, which has very few joints. Gypsum nodules are present in the rock mass and can act as nuclei for induced fracture growth.

The overbreak in the Rochester Formation was limited to the crown and invert, with no sidewall damage observed. In the haunch area, high angle induced fractures were observed to cut across the bedding creating slabs of rock and a stepped edge to the over break profile as shown in Figure 2a. A flat back was created by horizontal tensile fracture growth parallel to the bedding. The competent Lockport and Irondequoit limestones above and below the Rochester, respectively, and the low ground cover minimized the extent of the overbreak in this unit.

The Neagha is a fissile green shale, roughly 1.8 m in thickness and is the weakest unit crossed by the tunnel. The bedding thickness is less than 1mm and the fissility leads to almost zero tensile strength. The Neagha over broke, back to the overlying Reynales limestone, when exposed at the back of the roof shield of the TBM (Fig. 2b). The unit is extremely weak prior to excavation and the observed overbreak was related to flexural bending and gravity fall out. The competent units above and below the Neagha controlled the depth and width of the overbreak area.

The Grimsby Formation is irregularly bedded sandstone with dark red shale interbeds (Haimson 1983). Cross bedding is a common feature with in the Grimsby and being predominately competent sandstone, the overbreak was minimal. Minor loosening and bedding parallel fractures were observed to open 1-2 mm in the haunch area and fall out only occurred along thick shale layers (0.1 to 0.2 m) as they approached the tunnel crown, as seen in Figure 2c below. The weak shale layer promoted bed parallel fractures to develop, but the competent sandstone layers minimized the depth of overbreak.

The Power Glen Formation can be divided into two units. The upper unit is a light grey sandstone with grey interbeds of shale and the lower unit is predominately grey shale with interbeds of light grey sandstone. The upper unit had minor instability and dilation of beds in the crown, with minor overbreak along shale beds occurring, similar to the Grimsby Formation. The lower unit had significant overbreak in the haunch area, but was of limited height vertically above the crown due to the more competent upper unit. The width of the overbreak was also controlled by the stiff Whirlpool sandstone below, which reduced flexural bending of the beds in the lower Power Glen unit and minimized the tensile failure and overall overbreak dimensions.

The overbreak in the units above the Queenston Formation, (zone 1, Fig. 1) were influenced by the interbedded nature and the units above and below. Aside from added influences as mentioned above, the overbreak in zone 1 (Fig. 1) was controlled by the high horizontal in-situ stresses and the anisotropic nature of the rock mass. The overbreak in the Queenston Formation was influenced by the physiographic features, namely St. David’s Buried gorge (zone 3, Fig. 1).
and only locally controlled by other formations at the Whirlpool – Queenston contact (zone 2, Fig. 1).

The contact between the Whirlpool and Queenston Formations is a disconformity, an erosional surface parallel to the formation bedding, marking the transition from Ordovician (Queenston) to Silurian (Whirlpool) time. The erosional surface represents a gap in deposition when weathering, uplift and other degradation processes were occurring. The stiffness contrast between the Whirlpool and the Queenston creates a local stress shadow below the contact reducing the stress levels slightly. The stress in combination with local jointing and the presence of the strong Whirlpool above influenced the overbreak size and shape in zone 2 (Fig. 1). The overbreak was observed to break back to the overlying Whirlpool Formation to a maximum depth of 1.4 m at which time forward spiling was used to control overbreak and advance the tunnel. Overbreak approaching St. David’s Buried Gorge was limited due to the modified stress field, from the physiographic features, and once reaching the structural influence zone of St. David’s Buried Gorge overbreak reached depths in the order of three metres.

As the tunnel advanced away from the influence of St. David’s Buried Gorge the regional high horizontal stresses were encountered and overbreak continued to be greater than two metres in depth at the crown. The overbreak zone was characterized with steep sides where induced tensile fracturing was observed in the upper haunch area, with horizontal induced fracturing above the crown elevation, creating a plane dipping towards the face, likely due to stress rotation near the excavation face. A consistent notch shape (Fig. 2d), skewed to the left was observed likely indicating a high stress ratio with the major principle stress orientation slightly inclined from horizontal.

Sidewall tensile fracturing also occurred and was observed only on the left hand side wall likely indicating that the horizontal intermediate principle stress, $\sigma_2$, was sub-parallel to the tunneling direction. The localization of the tensile fractures to the left hand side only is likely the result of a low confinement zone on the left hand side as the intermediate stress flows around the excavation. It is likely that low confinement promotes tensile fracturing parallel to the minor principle stress, $\sigma_3$.

The variable ground conditions at the Niagara Tunnel Project raise the question, when do laminations within the rock mass become an important consideration for design purposes? How does the variation in lamination thickness affect the stability of the excavation? The units above the Queenston Formation range in thickness from 2 to 20 m and the stability of the excavation appears to be related to not only the units at the excavation face, but also to those above and below the tunnel. The Queenston Formation is believed to be over 300 m thick and the Niagara Tunnel is excavated into roughly the top 60 m. This provides an opportunity to study an anisotropic rock mass outside the influence of other formations. The numerical modeling presented in the following section will address these questions for a single intact rock type, which represents a clay shale.

3 MODELLING ANISOTROPIC GROUND CONDITIONS

Numerically there are several failure criteria in use for joints, for example Barton-Bandis, and Mohr-Coulomb models are used in Phase2, by Rocscience (2008). The most commonly used model is Mohr-Coulomb, which relates to the strength of a particular orientated plane to shear and normal stress limits. Much work has been conducted to determine the strength of a sample when a load is applied at a certain angle to the fabric of the rock. This was first proposed by Jaeger in 1960 and has since been updated (Jaeger et al 2007).

Friction and cohesion are important parameters to describe the failure along a plane of weakness. A lamination which is essentially an intact fabric will have values of friction and cohesion which are close to that of the intact rock, where as a continuous open joint (no rock bridges) will have considerably less strength. A true joint lies somewhere in between and will have portions of intact rock connecting the blocks and portions of open fracture. The resistance to slip is a function of both the rock and joint strength parameters (Chen et al 2007).
3.1 Model Setup

By modelling horizontal laminations with joint elements the rock mass behaviour is now controlled by both the intact rock properties between the joints and the joint properties themselves. The joint elements reduce the rock mass modulus in the vertical direction and allow for greater joint parallel displacements and for beam deflections, similar to that of a laminated voussoir beam.

For the purposes of this paper the intact rock properties, the geometry of the excavation and the joint properties have been fixed, so that a clear understanding of the influence of increasing lamination thickness on the excavation behavior can be determined. The model parameters used are listed in Table 1a below for reference and the various model types used are in Table 1b.

Dilation has not been included in the modeling and all materials are perfectly plastic. The material between the joints was assumed to have a GSI = 70, representing a material with some defects. Equation 1 below was used to relate the non-jointed models to the jointed models.
Table 1: a) Range of variables for numerical modeling

<table>
<thead>
<tr>
<th>Property</th>
<th>Values</th>
<th>Units</th>
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</thead>
<tbody>
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<td>Diameter</td>
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<td>m</td>
</tr>
<tr>
<td>Thickness</td>
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<td>m</td>
</tr>
<tr>
<td>UCS</td>
<td>40</td>
<td>MPa</td>
</tr>
<tr>
<td>Young’s Modulus, Ei</td>
<td>4</td>
<td>GPa</td>
</tr>
<tr>
<td>Depth</td>
<td>150</td>
<td>m</td>
</tr>
<tr>
<td>Ko</td>
<td>3</td>
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<tr>
<td>K_N</td>
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<td>MPa/m</td>
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<tr>
<td>Joint Tension</td>
<td>0.3</td>
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<td>Joint Cohesion</td>
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<td>MPa</td>
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<tr>
<td>Joint Friction</td>
<td>25</td>
<td>Deg</td>
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</table>

b) Model types

<table>
<thead>
<tr>
<th>Model Types</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Elastic with no joints</td>
</tr>
<tr>
<td>2. Plastic with no joints</td>
</tr>
<tr>
<td>3. Rock elastic and joints elastic</td>
</tr>
<tr>
<td>4. Rock plastic and joints elastic</td>
</tr>
<tr>
<td>5. Rock plastic and joints plastic</td>
</tr>
<tr>
<td>6. Transversely isotropic elastic</td>
</tr>
</tbody>
</table>

\[
\frac{1}{E_{rm}} = \frac{1}{E_i} + \frac{1}{K_N T}
\]

Equation 1: Rock mass modulus, \( E_{rm} \), as a function of intact modulus, \( E_i \), and joint spacing, \( T \). Relationship for a transversely isotropic material (Brady & Brown 2006).

Using Equation 1 the rock mass modulus was determined accounting for the decreased stiffness due to the presence of the joints. Using the intact properties the GSI value was adjusted in RocLab, by Rocscience, until the deformation modulus from RocLab was equal to \( E_{rm} \) from Equation 1. The rock mass strength parameters had to be scaled also to reflect the intact material properties. This was done using harmonic averaging, similar to that for defining the hydraulic radius of an excavation. This ensured that the stiffness and strength parameters were reduced such that the non-jointed models reflected the equivalent jointed models.

3.2 Model Validation

Phase2 is a finite-element continuum code with integrated Goodman (Goodman et al. 1968) joint elements. Discontinuum modeling software UDEC can also be used to represent jointed rock masses. The discontinuum models allow for large strains, block rotation, detachment, and new contact formation. UDEC uses a time stepping mechanism to solve for equations of motion (Itasca 2000), which allows for greater displacements than the continuum models of Phase2, which do not allow movement after detachment and prohibits new contact formation.

To validate the model behaviour the non-jointed and jointed models were computed with both UDEC and Phase2. The jointed model results for both UDEC and Phase2, for various lamination thicknesses, give similar yield zones around the excavations, as compared in Figure 3 below. For comparison the jointed models in Figure 3 are also compared to the non-jointed models. There is a substantial difference in the yield zone size and shape between the non-jointed and jointed models. This illustrates the importance of anisotropic behaviour which has been induced by the joint elements in the models.

UDEC also offers a ubiquitous joint model, which allows for failure on oriented weak planes as well as in the rock, but does not model an explicit location of the weak planes (Itasca 2000). By allowing failure on a weakness plane to be checked within every zone of the model, the rock mass strength is essentially reduced to the weak plane criteria when the stresses are orientated correctly. The ubiquitous joint model will overestimate the plastic yield zone when the lamination thickness is a measurable quantity because the material in between the laminations is much stronger than the laminations themselves.
Phase2 does offer a post-analysis ubiquitous joint strength factor calculation. This does not allow for progressive deformation on an orientated weak plane during computation and is therefore not compared to the UDEC Ubiquitous joint model which does allow progressive deformation after slip (but not in an oriented manner). The ubiquitous joint feature in Phase2 allows for plotting of strength factor contours accounting for an orientated ubiquitous weak plane (Rocsscience 2008).

Since the plastic yield zone for both the UDEC and Phase2 models are similar the remaining discussion on crown deflections and the excavation behaviour styles with decreasing lamination thickness will be based on Phase2 results only.

3.3 Influence of lamination thickness on excavation behaviour

Rock mass displacements are an important factor to consider when designing rock support systems. For horizontally laminated rock masses, these displacements in the tunnel crown are best described as bed deflections. The classic voussoir analysis for a laminated elastic beam, presented by Diederichs and Kaiser (1999), is a method for calculating mid beam deflections and stresses for a rectangular excavation. Other analytical solutions exist for elastic isotropic displacements around circular openings and can be found in most rock mechanics text books, for example in Brady & Brown (2006). Anisotropic elastic solutions also exist for simplified cases such as a transversely isotropic or an orthotropic material. These simplifications reduce the required elastic constants of the stress-strain tensors and make a closed form solution possible. However, in practice it is difficult to determine all the elastic constants for an engineering project and these solutions are rarely used for design purposes (Brady & Brown 2006). These methods have been found to under estimate the crown deflections as presented in Figure 4 below.
Six different model types were used to determine the influence of laminations on the rock mass response. The isotropic elastic model with no joints represents the base case. Elastic models are used to determine the maximum stress concentrations around the excavation. Since the material can not yield plastically the displacements are not realistic. By including the laminations in the elastic model (rock and joints elastic) the change in rock mass stiffness due to the joint stiffness contribution increases the deflections from those found in the isotropic elastic case, but these deflections remain low since the material can not yield (Fig. 4). The transversely isotropic model results are the same as the rock and joint elastic model indicating the accuracy of the transversely isotropic model. Treating the rock mass as an equivalent isotropic material and reducing the intact properties to account for jointing and other heterogeneities is standard engineering practice. This allows for yielding in the rock mass and increases the displacements over the elastic models, however the crown deflections still remain lower then the rock and joint plastic model. It is interesting to see (Fig. 4) that by including the laminations, but not allowing slip (joints are elastic) that the crown deflections are similar to the equivalent isotropic plastic model results. This indicates the importance of joint slip on the excavation behaviour. By allowing slip (joints plastic) the excavation response to decreasing lamination thickness can be broken down into 5 sections as indicated in Figure 4.

When the lamination thickness is on the order of the excavation radius, then an equivalent isotropic rock mass is found to satisfactorily represent the laminated rock mass. In section 1 of Figure 4 the non-jointed model crown deflections are similar to those of the jointed model. Also the plastic yield zone (Fig. 3) is similar between the two model types. It is characterized as a dome shape of roughly 2.5 metres high. There is some truncation of the yield zone in the jointed model in the haunch area. The specific location of the lamination with respect to the tunnel will control the extent of plastic yield and rock support should target the specific laminations.
As the lamination thickness decreases the stresses begin to channel between the crown and the first lamination, which begins to occur at a thickness of 6 meter in Figure 4. The stress channeling increases the stress through the first beam above the crown causing increased deflections, as seen in section 2 of Figure 4. Again the plastic yield zone shape is controlled by the specific location of the lamination. Note the shear failure underneath the lamination cutting through the haunch area in Figure 3, section 4 indicating that lamination slip is beginning to play an important role in the excavation stability. In the modelling presented here the distance up from the crown to the first lamination has been set to half the lamination thickness, there for laminations with in 3 - 4 metres above the crown of the excavation can begin channel the stresses and increase the crown deflections. Rock support should continue to target specific laminations to minimize slip so that the stress can be re-distributed into the rock mass.

At a lamination thickness of 2.4 metres a second lamination unit becomes involved. Below 1m, the stresses are distributed over multiple laminations and the crown deflections follow a similar trend as the voussoir model predicts (Diederichs & Kaiser 1999). The deviation from the classic voussoir, which is plotted in Figure 4 as the dashed line, is due to the high stresses and the circular excavation. Section 3 of Figure 4 also marks the start of tensile failure in the haunch area around the excavation. Plastic yielding and lamination slip now extends several metres above the excavation and is not truncated by the presence of the laminations. The shape of the yield zone has near vertical sides above the crown and joint slip extends well beyond the excavation (Fig. 3). Rock support should now focus on tying the laminations together to create a composite beam as suggested by Lang et al (1979).

In section 4 of Figure 5 the degree of tensile failure has increased and extended into the laminations above the crown. This is indicative of crown beam failure and the voussoir predicts snap through of the laminated beam at 0.9 metres. However the circular excavation geometry provides additional support that is not accounted for by the voussoir model and the crown deflections do not increase as rapidly as the voussoir predicts. The geometry of the plastic yield zone in section 4 is characterized with sub vertical sides and ‘wings’ extend out from the haunch area. A high stress confinement area between the crown yield area and the haunch yield area, as indicated in Figure 3, exists. This high confinement area is sensitive to stress changes induced by rock fall or near by excavation advance which will redistribute the stress field and may result in more plastic yielding.

At 0.4 metres, in Figure 5, the extent of plastic yield has stabilized and reducing the lamination thickness only has an elastic affect on the rock mass. Note the slope of the plastic jointed model in section 5 of Figure 5 and the slope of the rock elastic, joints elastic model. The degree of plastic yielding has damaged the rock mass in the yield zone to a point which has created a new elastic material.

3.4 Anisotropic Rock Mass Behaviour

The deformation response of a large scale circular excavation in anisotropic ground conditions is substantially different then that of an equivalent isotropic rock mass. The state of practice in engineering design is to treat the rock mass as an isotropic medium, reducing the intact modulus to a deformation modulus through various techniques to compensate for structure, microscopic defects and other material heterogeneity. Generally this works well in most cases where the geometry of the structure creates an interlocking block assemblage. Testing by McLamore and Gray (1967) presented by Hoek and Brown (1982) show that as the number of joint sets increase, the rock mass behaves more and more isotropically. If the discontinuity spacing is on the order of the excavation diameter, then the engineer must deal with these discontinuities specifically, either numerically or analytically, as in wedge analysis. When the structure or laminations within the rock mass are parallel with few interconnections, the spacing is small and the intact modulus is low, then the rock mass behaviour will be distinctly different then the isotropic model predictions. The failure mechanism is controlled by lamination parallel displacements and lamination deflections. These two motions are not accounted for by equivalent isotropic models, where the displacements are radial and no laminations exist to deflect.
The high lateral displacements and bending of the beds in the haunch area promotes tensile failure of the rock mass. As the lamination thickness decreases more tensile failure occurs in the haunch area which promotes greater deflections of the beds above the crown. As the lamination thickness decreases further, the tensile failure extends above the crown, until a stabilized plastic yield zone is achieved at low lamination thicknesses.

4 CONCLUSIONS AND IMPLICATIONS

Anisotropic rock mass behaviour is controlled by weakness planes within the rock mass, which can be joints, bedding, cleavage or other parallel features which give rise to the preferential plane of weakness. The strength of the weakness plane and the intact rock determine if the rock mass will behave anisotropically and limitations of these properties in the literature are not presented for excavation design. The state of practice in excavation support design is to treat the rock mass as an equivalent isotropic material, reducing the intact strength properties to account for heterogeneity. By including the laminations in the numerical modeling presented, the anisotropic behaviour has been studied and significant differences have been found to exist from the equivalent isotropic model.

Five different excavation behavioral areas have been found to exist as the lamination thickness is decreased. For the 16m tunnel investigated, where spacing is greater then 6 metres, the isotropic model and the anisotropic model give the same crown deflections and the yield zone is similar. Between 2.4 and 6 meter thicknesses, stress channelling between the crown and the first lamination above the crown causes higher stress concentrations and increased deflections. Below 2.4 metres, a second lamination becomes involved and the behavior resembles classic voussoir instability. A thicknesses below 1m (for a 16m excavation), the stresses flow through multiple beams and the behaviour is analogous to a laminated voussoir beam with variations accounting for the circular geometry. This behaviour continues past the point at which the voussoir model predicts snap through failure at 0.9 metres. The haunch rock mass provides additional support, which prevents rapid deterioration of stability. At 0.4 metres the plastic yielding reaches a self limiting state.

By including the laminations in the numerical modelling substantial differences in the plastic yield zone size and shape and the magnitude of the crown deflections are found to exist. Although a detailed parametric study has not been conducted, the results do indicate that even at large lamination spacings the variation from the equivalent isotropic model results can be significant. Some suggested considerations for rock support design based on the findings of this study follow.

The size and shape of the yield zone above the tunnel crown provides a location of undamaged rock 1 – 2 metres back from the haunch area and is a zone of high confinement which prevents fracture growth. The orientation of rock bolts should utilize this undamaged area. The high angled sides of the yield zone do not provide a stable abutment geometry if the haunch rock mass, which provides support for the beds above the crown, fall out during advance. By using C-channels or other strapping, the rock bolts and rock mass are knit together forming an arch for support. Installation as close to the tunnel face as possible should be utilized for all rock support, which will prevent slip on the weakness planes.

The lateral displacements below the crown elevation are important to the failure mechanism and by installing rock support to reduce the amount of lateral slip the yield will also be reduced, both in the haunch area and above the crown.

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