Tunnelling in horizontally laminated ground

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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*

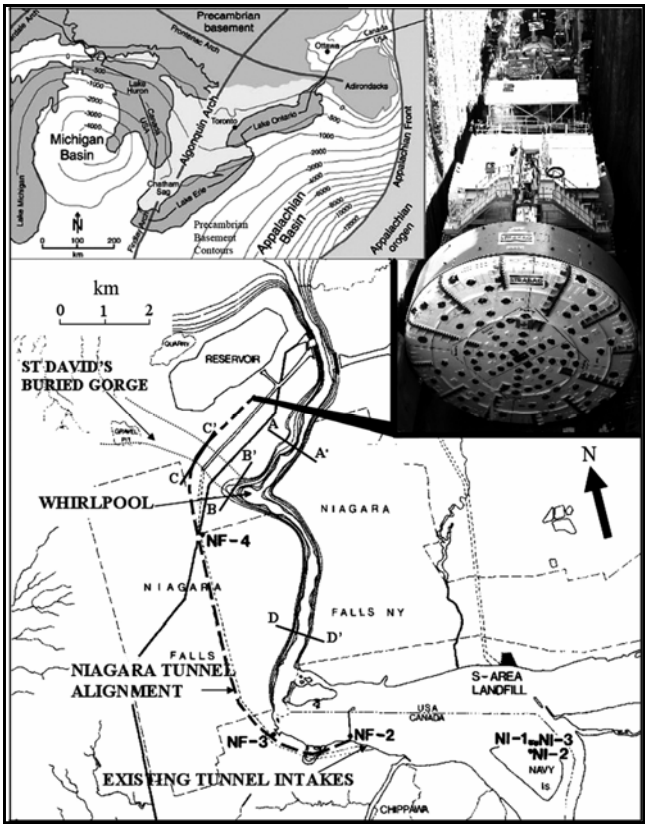
The Appalachian Basin and Arch influenced the deposition of the Ordovician and Silurian strata of the Niagara Region. The Appalachian Orogen provided clastic sediments and variations in sea levels controlled the depositional environment (Mazurek 2004); consequently, the strata in the Niagara Region vary 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).



Age	Group	Formation	Brief Description	Approx. Thickness (m)	Symbol
Middle Silurian	Albemarle	Lockport	Grey crystalline dolomitic Limestone	16.8 – 20.3	
		Decew	Crystalline dolomite &grey mudstone	2.1 – 4.0	
	Clinton	Rochester	Dark grey calcareous shale dolomite interbedded	17.7	
		Irondequoit	Grey to reddish dolomitic limestone	1.2 – 3.1	
		Reynales	Light grey crystalline dolomite	3.6	
		Neahga	Green shale	1.8	
		Thorold	White sandstone	2.4	
Lower Silurian	Cataract	Grimsby	Green, irregularly bedded sandstone with red shale interbeds	16	
		Power Glen	Grey shale to white calcareous sandstone	11.5	
		Whirlpool	Light grey crossbedded sandstone	3.6 – 7.6	
Ordovician		Queenston	Red shale and argillaceous limestone	335	

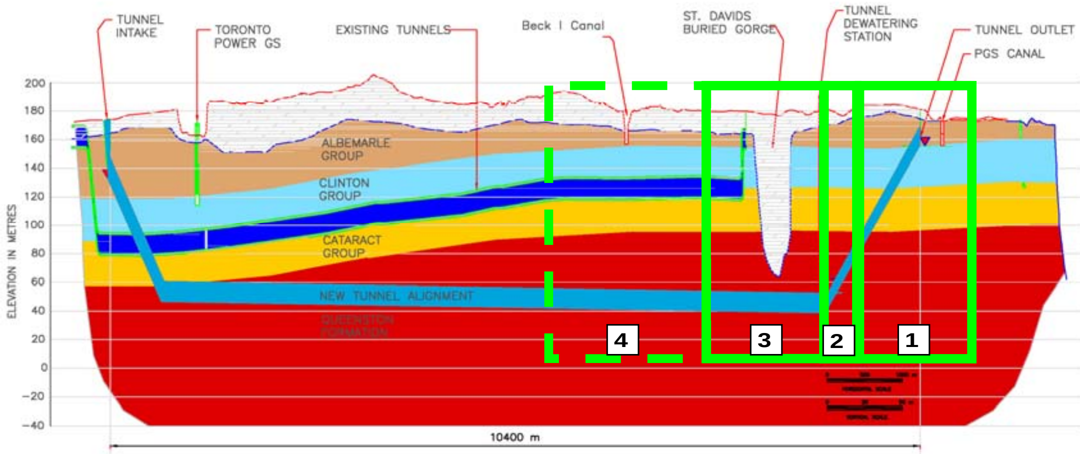


Figure 1: Physiographic features of the Niagara Region (modified from Novakowski and Lapcevic 1988), with inset map showing regional setting (Mazurek, 2004), and photo of TBM ready for launch (courtesy of OPG). Geological stratigraphy on right colour coded to match Niagara Tunnel Cross Section below.

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, Ko, 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, σ_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, σ_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 (Jaegar 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).

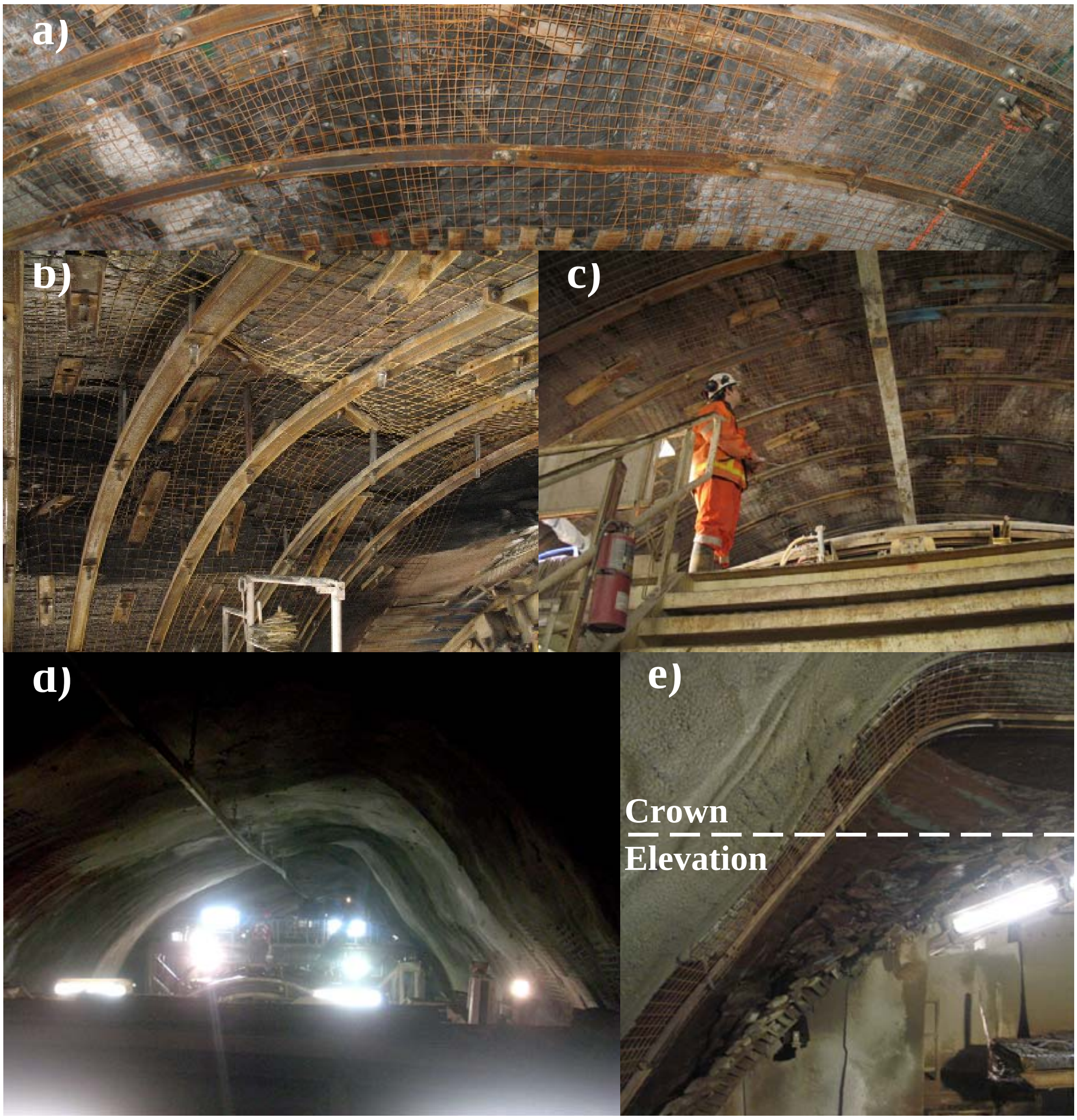


Figure 2: Photos courtesy of Ontario Power Generation Inc showing overbreak in the Niagara Tunnel Project a) Narrow width in the Rochester Formation b) Haunch only in Power Glen Formation lower unit c) Shallow along 0.2m thick shale beds, dilation d) Stress induced notch e) Left haunch showing vertical tensile fracturing (bottom) rotating to horizontal above the crown elevation.

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

Property	Values	Units
Diameter	16	m
Thickness	0.16 to 8.0	m
UCS	40	MPa
Young's Modulus, Ei	4	GPa
Depth	150	m
K_o	3	-
K_N	25000	MPa/m
K_S	2 500	MPa/m
Joint Tension	0.3	MPa
Joint Cohesion	0.14	MPa
Joint Friction	25	Deg

b) Model types

Model Types
1. Elastic with no joints
2. Plastic with no joints
3. Rock elastic and joints elastic
4. Rock plastic and joints elastic
5. Rock plastic and joints plastic
6. Transversely isotropic elastic

$$\frac{1}{Erm} = \frac{1}{Ei} + \frac{1}{K_N T} \qquad \textbf{(1)}$$

Equation 1: Rock mass modulus, $E_{rm,}$ as a function of intact modulus, E_i, and joint spacing, T. Relationship for a trasversely 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 then 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 then the laminations themselves.

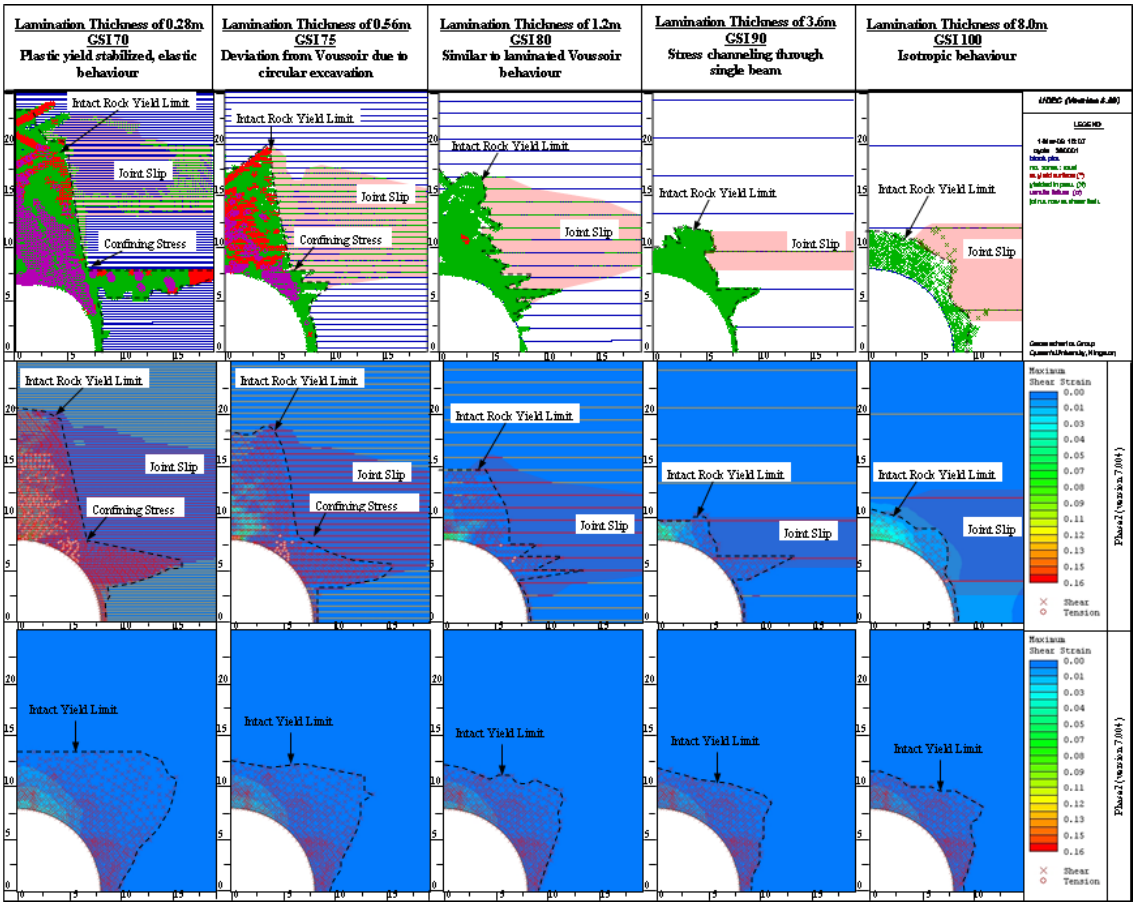


Figure 3: Yield zones for various lamination thicknesses, comparing non-jointed and jointed models from Phase2 and jointed models from UDEC.

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 there for 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 (Rocscience 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 import 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.

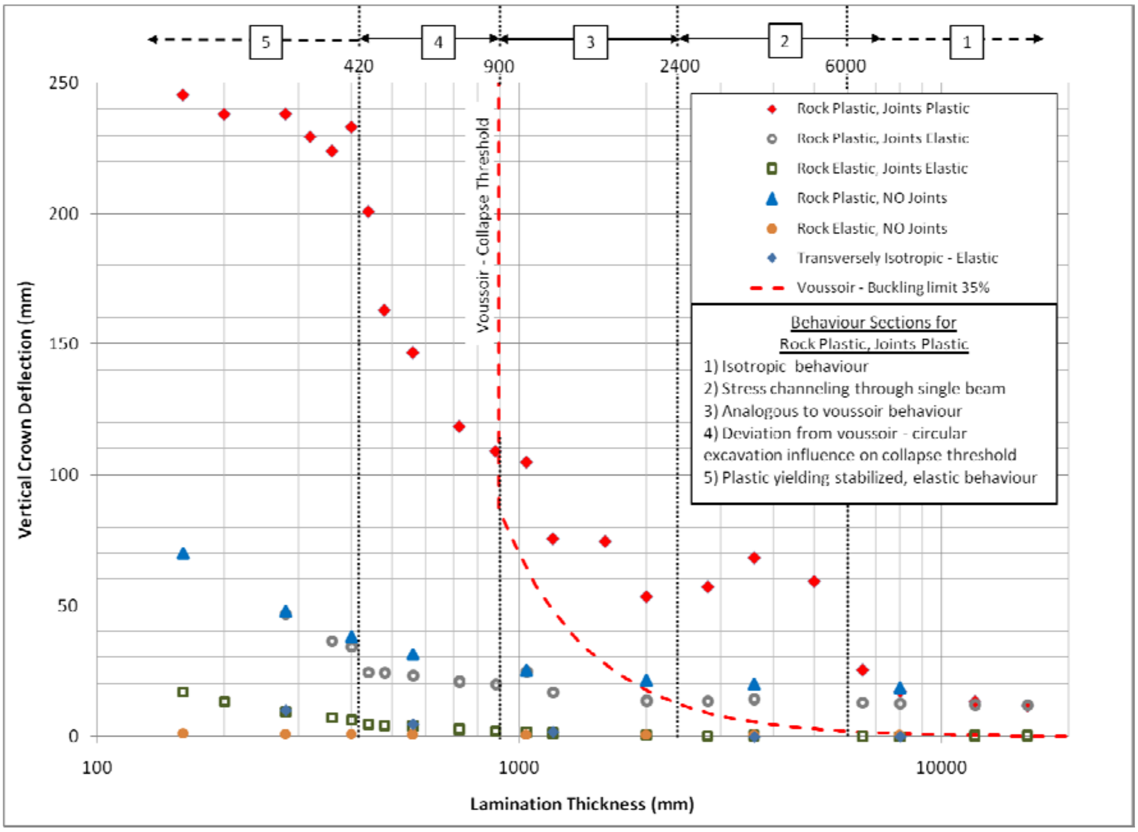


Figure 4: Crown displacements with respect to lamination thickness for Phase2 models and Voussoir mid span deflection results based on Diederichs and Kaiser (1999). Note the five different excavation behaviours with increasing lamination thickness.

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 analsysis. 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.

The high confinement zone mentioned above could lead to further collapse if disturbed during scaling or if the stress field is modified by a nearby excavation. The high confinement zone minimizes the fracture growth in this area and if the stresses around the excavation are modified allowing fracturing to develop the effective span could be increased. By increasing the effective span the width of the yield zone above the crown will also increase, thereby increasing loads on existing rock support and potentially allowing more overbreak to collapse into the tunnel. These key factors should be considered when designing support and implementing installation to minimize and prevent further overbreak.

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