Influence of Rock Mass Anisotropy on Tunnel Stability

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ABSTRACT: Recent works in deep underground tunnelling have illustrated how the behaviour of the ground can differ from those anticipated (Kaiser 2005 & 2006). Two reasons for this deviation are: (1) effect of structural features such as joints, bedding planes or foliations, facilitating the stress-driven rock mass disintegration process; and (2) rock mass degradation as a result of stress damage processes that were not properly taken into account in the lowering of rock mass quality of the near excavation boundary rock. In this paper, rock mass behaviour aspects affecting design and the use of simple tools to assess the influence of structural features on excavation stability are discussed. From this work, an Orientation Reduction Factor (ORF) based on a modification to the Stress Reduction Factor (SRF) developed by Barton et al. (1974) is proposed to assist in the design of tunnels in anisotropic rock.

1 INTRODUCTION

Stress induced, gravity-assisted failure processes typically dominate long before stress levels approach the strength of the intact rock (at approximately $\sigma_{\text{max}} > 0.4$UCS where $\sigma_{\text{max}} = 3\sigma_1 - \sigma_3$ and UCS is the Unconfined Compressive Strength of the intact rock (Martin et al. 1999)). Under these conditions, discontinuities typically become clamped and the failure process becomes brittle. This brittle failure process is dominated by new stress induced fractures growing subparallel to the excavation boundary. As the stress induced fractures accumulate, the rock quality surrounding the near boundary of the opening is degraded causing dilation (i.e. volume increase) which results from three potential sources:

− Dilation due to tensile fracture initiation and growth;
− Shear along existing fractures or joints; and
− Bulking due to geometric incompatibilities that are created when blocks of broken rock move relative to each other as they are forced into the excavation.

Rock mass bulking produces large, permanent, radial deformations in the fracture zone and consequently at the excavation wall (Kaiser et al., 1996). In addition, as the rock mass degrades, the strength of the damaged rock reduces and larger rock blocks are broken in to smaller blocks. Thus the rock quality of the near boundary rock material is also reduced. The depth to which the near boundary rock is influenced by stress induced damage can be estimated for excavations in brittle failing rock using the following formula (Kaiser et al., 1996):
where: $\sigma_{\text{max}} = 3\sigma_1 - \sigma_3$; $\sigma_c = \text{unconfined compressive strength of intact rock (laboratory peak strength)}$; $a = \text{tunnel radius in meters or radius of circumscribed circle to the excavation}$; $d_f = \text{depth of failure in meters measured from the circular boundary}$.

The degradation process is summarised on Figure 1 which illustrates how the rock mass directly surrounding the excavation boundary has been characterized ‘pre’ degradation with a Geological Strength Index (GSI) value generally >70 and ‘post’ degradation with GSI ranging from 35 to 50 based on field observations (Kaiser, 2006). The resulting behaviour of the ‘post’ excavation rock mass will be different from the ‘pre’ excavation assessed behaviour. This has a drastic effect on constructability as the stand-up time for the degraded rock near the boundary of the opening is reduced.

According to Kaiser (2006) this can cause:

- Unexpected deep-seated unravelling even if the in situ rock mass is massive to moderately jointed;
- Impractical use of standard bolting due to the small block size in the disintegrated rock mass area;
- TBM gripper stability problems due to unexpected failures along the excavation walls; and
- Unexpected floor heave.

While it is possible to identify areas with the potential for brittle failure, for example, using the general assumption that brittle failure will occur in areas at right angles to the major principal stress (Hoek et al., 1995), when the threshold $\sigma_{\text{max}} > 0.4 \text{UCS}$ is reached in areas around the tunnel boundary, or elastic modelling and the Hoek-Brown brittle parameters proposed by Martin et al., (1999) (i.e. $m \approx 0$ and $s \approx 0.11$), amongst others, recent observations discussed by Everitt & Lajtai (2004), and Kaiser (2005; 2006) suggest that brittle failure processes are en-
hanced when structural features such as joints, weakness zones, bedding planes or foliations are preferentially orientated around the excavation boundary creating stress raisers which in turn facilitate the stress-driven rock mass disintegration process (e.g., as illustrated by Everitt & Lajtai (2004) on Figure 2b).

Such weaknesses induce stress heterogeneities that thus promote tensile-spalling failure processes (Diederichs 2000). The observations suggest that stress induced damage can occur “prematurely” depending on the orientation of micro and macro structures in situations where the known stress conditions should not yet initiate brittle failure processes. In general, when structural features (micro and/or macro), especially foliations, are present in a rock mass, the strength is effectively reduced, largely due to tensile strength heterogeneity in areas depending on the orientation of the structural features with respect to the excavation boundary. As a result, stress induced damage could occur in areas not normally considered due to the influence of anisotropy (Figure 2).

Figure 2: a) URL test tunnel reproduced after Martin et al., (1999) showing stress as dominating control on location of stress induced damage. b) Interpretation by Everitt & Lajtai (2004) showing foliation orientation as dominating control on location of stress induced damage. c) & d) breakouts observed in the tunnel back in Leventina-Gneiss along foliation planes with a $k_e=0.7$ where failure should have been in the walls if stress levels were sufficient (Kaiser, 2006).
1.1 Rock Mass Classification and Characterization

Current classification approaches, empirical design methods, and rock mass parameter estimation techniques such as the Rock Quality Index Q system (Barton et al., 1974); the Rock Mass Rating system (RMR) (Bieniawski, 1973, 1989); and the Geological Strength Index (GSI) (Hoek et al., 1995) do not specifically take into consideration the ‘post’ excavation rock mass degradation in brittle rocks and certainly not the affects of anisotropic behaviour due to the orientation of weakness planes (foliation) and structural features. Fundamentally, most systems implicitly assume rock mass homogeneity and isotropy.

A factor to account for the effects of structural features under moderate to high stress levels on excavation stability is needed to enhance the utility of empirical design methods. For anisotropic rock masses, Marinos et al., (2007) discuss how the stress-dependent regime is controlled by the anisotropy of the confined rock mass and points out that the main classification systems (i.e. Q, RMR and GSI) do not properly account for rock mass behaviour and strength reductions as a result of the directionality/anisotropic heterogeneous nature of the rock mass with respect to loading. Marinos et al., (2007) suggest that in cases where anisotropic rock mass behaviour is present, it would be necessary to develop an orientation-dependent rock strength based on an orientation dependent UCS. Accordingly, the GSI value would remain high and the rock mass strength would be determined by the orientation-dependent UCS value. Alternatively, an orientation dependent GSI with a UCS that is representative of the inter-structural intact rock strength could be used to achieve the same effect.

As the above methods are also used for support selection (namely the Q system), ignoring the lower rock mass strength due to preferentially oriented structural features will lead to inappropriate support designs (increasing project costs) because the dominating rock mass behaviour mechanisms are not considered (Kaiser 2005 & 2006).

2 INFLUENCE OF STRUCTURAL ORIENTATION ON TUNNEL BEHAVIOUR

Anisotropy in a rock mass resulting from preferentially orientated structural features (micro and macro scale) around the boundary of an underground opening affects the extent and shape of low confinement zones around the boundary of an excavation (see Bewick & Kaiser, 2009 for open stope example and Bewick, 2008) and therefore directly influences the location and depth of stress driven rock mass degradation which is a tensile driven process.

When structural features (micro and/or macro), especially foliations are present in a rock mass, the strength is effectively reduced largely due to tensile strength heterogeneity depending on the orientation of structural features with respect to the excavation boundary (i.e. loading direction relative to the anisotropy). In general, anisotropy:

− Affects the strength of the rock mass depending on loading direction;
− Causes a non-uniform stress state and displacement around the boundary of an underground opening;
− Creates a non-uniform depth of failure; and
− Allows for stress induced failure (near wall degradation) to possibly occur at relatively low stress levels (Everitt & Lajati, 2004) and under unexpected stress conditions.

These conditions may lead to construction problems due to geometric rock mass bulking and unravelling ground (i.e. requiring deformation based design and consideration of reduced stand-up times) (Kaiser 2006).

Anisotropy in a rock mass, resulting from preferentially orientated structural features (micro and macro scale), affects the extent and shape of the low confinement zones and therefore directly influences the location and depth of stress driven rock mass degradation.

As a result, stress induced damage could occur in areas not normally considered due to the influence of anisotropy creating variations in the depth, location, and overall amount of damage. Therefore, stress induced damage is not just a function of stress but a function of stress and rock mass strength anisotropy. These observations are supported by field evidence recorded by Brox & Hagedorn (1996), Everitt & Lajtai (2004), and Kaiser (2006).
This is illustrated on Figure 3 for some idealistic bedded rock mass geometries (Phase 2 model of circular tunnel in bedded rock mass), showing the location of rock mass damage as being highly dependent on the orientation of anisotropy and less dependent on in situ stress ratio. This is in contrast to Martin et al. (1997) where the depth of failure was assumed to be only a function of stress \((\sigma_{\text{max}})\) and intact rock strength \((\sigma_c)\). The influence of anisotropy (geologic heterogeneity) was already introduced as a primary factor by Everitt & Lajtai (2004) possibly explaining the reason for the non-symmetry of the URL stress induced breakouts.

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\begin{align*}
  k_o &= 0.5 \\
  k_o &= 1.0 \\
  k_o &= 1.35
\end{align*}
\]

![Figure 3](image_url)

Figure 3: Example of tunnel in rock mass with single joint set showing \(\sigma_3\) contours showing that areas of lower confinement are more controlled by structure orientation rather than stress ratio.

3 INFLUENCE OF STRUCTURE ON STABILITY OF CIRCULAR TUNNELS

A parametric study was completed using the program Phase 2 (©RocScience 2006) to assess the influence of the orientation of a dominant structural set on a 10 m diameter circular tunnel. A discussion on the material properties used in the models and the details of related sensitivity analyses can be found in Bewick (2008). The results for low confinement and tensile rock mass failure are presented on Figure 5 for all assessments (i.e. varying joint friction angle, in situ stress ratio, joint element spacing, etc.).

A typical example showing the influence of a dominant structure is shown on Figure 4. The relative orientation \((\beta)\) of the single joint set around the boundary of the circular tunnel influences the state for stress creating zones of low joint confinement in characteristic areas around the tunnel boundary. These areas are located in an approximate 100° window, 50° on either side of a direction perpendicular (i.e. \(\beta\) angles 0° to 50°) to the jointing. In this window, the
weakness planes are critically aligned in the direction of tangential stress and thus are allowed to shear. As a result, the depth of low confinement is enhanced. In other directions, the structure is clamped and little to no shear movement is possible, thus restricting the confinement zone to those anticipated from analyses without weakness planes.

The depth of low confinement is strongly influenced by the yielding or slip of the joint elements (weakness planes), suggesting that as the joints or structures around the boundary of a circular opening mobilize, the local boundary stress state is perturbed. This in turn creates zones of low confinement that promote brittle failure mechanisms; crack initiation, coalescence and propagation. The shape and extent of a reduced confinement zone near the excavation also indicates that there is an inner shell (Kaiser & Kim 2008 a and b) where brittle, tensile failure processes preferentially occur. In this inner shell, ‘joint’ yield promotes rock mass degradation (yield). In the outer shell, where the rock mass is sufficiently confined, only ‘joint’ yield may occur as the confinement is high enough to prevent tensile crack-damage degradation (thus this is a zone of higher strength). Additional insight into the strength of the inner and outer rock mass shells can be found in Kaiser & Kim (2008a/2008b). When failure is predicted on joints and in the rock mass, this does not mean that the residual post-peak state is reached and rock disintegrates. However, when the confinement is low (such as in the inner shell), then rock can fail and disintegrate into smaller blocks (i.e. there is the potential for brittle failure process to dominate). This degradation process then causes frequent ground control and constructability problems, e.g., during TBM advance (Kaiser, 2007).

3.1 Parametric Assessment

A number of parameters were assessed to determine their influence on the effect of structure orientation \( (\beta) \) around the boundary of a circular tunnel with a single joint element set. Note that the word joint is used hence forth and is used to represent systematic foliation, layering, etc. Based on the parametric study, Bewick (2008) suggests that:

- The influence of structure is not only a function of relative orientation around the excavation boundary but also a function of the bedding thickness (i.e. spacing between structural features) and the strength of the intact rock (between the structures), both of which influence the
predicted depth and lateral extent of shear yielding, tensile failure and low confinement levels around an excavation;
- An excavation span to joint spacing ratio of \( \geq 10 \) was required for the structures to influence the depth of yield and low joint confinement zone around the tunnel. When the excavation span to joint spacing ratio was \(<10\), strength was mostly controlled by that of the intact rock;
- The depth of low joint confinement is mostly limited to approximately \( 0.2a \) where \( a \) is the tunnel radius;
- Up to a stress to strength ratio \( \sigma_{\text{max}}/\sigma_c \) of 0.8, the zones of joint yield, shear yield, tensile failure, and low joint confinement are controlled (dominated) by the relative orientation of the jointing around the tunnel boundary. Once the \( \sigma_{\text{max}}/\sigma_c \) ratio is \( >0.8 \), shear behaviour dominates and the influence of the structures is overshadowed by the high stress level relative to the strength of the rock mass between the weakness planes. It is thus anticipated that brittle failure processes will dominate at stress to strength ratios \( \sigma_{\text{max}}/\sigma_c \leq 0.8 \). At higher stress ratios, shear failure or squeezing behaviour must be anticipated; and
- Depending on these characteristics, the maximum depth of yield is approximately at the edge of the 50° window (i.e. 50° to the left or right in a direction normal to the parallel structure) rather than in the centre. The maximum depth of yield and thus rock degradation occurs at this point as a result of the geometry of the circular tunnel and parallel jointing. At this point a tight radius of curvature is created between the tunnel and structural feature allowing for stress induced fractures to propagate from the yielding structure. Locally, this results in a deeper low joint confinement zone.

4 PROPOSED TUNNEL ORIENTATION REDUCTION FACTOR (ORF)

Based on the observations and modelling results discussed above and examples analysed in detail by Bewick (2008), stress induced yield is observed to occur in areas that would not be identified using current stress induced stability analyses (i.e. Martin et al. 1999, Kaiser et al. 2000, and Diederichs, 2004) and occur even when the stress magnitudes should not mobilize the stress induced failure process in massive rock. Therefore, an Orientation Reduction Factor (ORF) is proposed to assist in identifying conditions where anisotropic strength or weaknesses may affect premature failure. The purpose of this factor is to allow for easy identification of areas where stress induced yield could potentially occur around a circular excavation in association with a persistent structural or foliation pattern even when the stress state would typically not allow for yield to develop. In this manner, it should be possible to better predict potential construction difficulties due to unravelling and stand-up time reductions in anisotropic ground.

Based on sensitivity studies (Bewick 2008), characteristic low confinement zones develop in areas around the circular tunnel due to the radial stress gradient and the effect of jointing near the excavation boundary (Figure 4). These low confinement zones are areas with a higher potential for stress driven rock mass failure. By plotting all the results for the depth of tensile failure and low confinement around the tunnel boundary (\( \beta = 0^\circ \) is at a location on the right sidewall at the springline in a direction normal to the vertical joint elements and \( \beta = 90^\circ \) a rotation counter clock wise from 0° - ref. Figure 5), the typical extent of the influence of the joints was obtained (Figure 5). From these figures, it can be seen that the typical depth of low confinement and tensile failure is approximately 1 to 1.25m, or 0.2a to 0.25a for this 10m diameter tunnel, and limited to a lateral extent of approximately 50° on either side of a direction normal to the structures.
As indicated above, joint orientation influences the confinement near the opening and thus the strength of the rock mass (i.e. as confinement increases strength increases). In addition, brittle failure is a tensile failure process and is mobilized in zones of low confinement. Strength increases and decreases as a result of confinement is quite evident from triaxial testing on intact rock samples. The influence of confinement in this manner can be extended to the excavation boundary such that, as a result of the low confinement zones that develop due to the radial stress gradient and the orientation of the jointing, it should be possible to modify the intact compressive strength of the rock and rock mass. Modifying the UCS value allows the influence of joint orientation around a circular tunnel to be based on the Stress Reduction Factor (SRF) proposed by Barton et al. (1974).

Figure 6 shows how the depth of yield around a circular tunnel with 10 m diameter in an isotropic homogeneous rock mass changes with decreasing UCS using the empirical equation proposed by Kaiser et al. (1996) and Martin et al. (1999). As can be seen from this figure, no yield is anticipated at the rock strength of 200 MPa by the empirical equation for the considered state of stress. Comparing the depths of yield from the empirical equation to the typical maximum depth of low confinement and tensile failure due to the preferentially oriented structures (i.e. approximately 1.25 m), it can be seen that the UCS would have to be effectively lowered from 200 MPa to approximately 125 MPa based on the empirical equation (arrows on Figure 6). This lower UCS values relates to percent change of approximately -38%, respectively. Comparing
the orientation based strength change obtained from laboratory UCS testing discussed by Everitt & Lajtai (2004), the peak strength with the foliation orientated 70° to the sample is 210MPa compared to the 120MPa strength that is obtained when the sample is loaded with the foliation orientated 10° to the sample or a percent change of approximately -43% which compares well with the percent change of strength obtained from the empirical results. This provides some confidence in the strength drop that appears to occur due to the orientation of persistent weakness features near the excavation boundary in the modelling using the assumed parameters discussed in Bewick (2008). It is important to note that this strength modification should only be used in a window of 50° on either side of a direction normal to preferentially orientated structural set.

![Figure 6: Influence of UCS on depth of yield.](image)

The effective lower UCS changes the $\sigma_{\text{max}}/\sigma_c$ ratio from approximately 0.4 (using the 200 MPa strength) to approximately 0.65 (125 MPa strength) based on the empirical equation. This represents a change from essentially elastic ground with a SRF of 1 to stress yielding ground with a SRF of ~25. For the model parameters used to assess the influence of structure orientation around the boundary of the tunnel, considering first the Q and then the GSI system, rock quality is reduced from Q’ ~54 (Very Good Quality) to an anisotropic Q, Q_{an} of ~2.2 (Poor Quality) and a GSI of ~80 to an anisotropic GSI_{an} of ~50. Considering Bieniawski’s stand-up time chart and converting GSI to RMR, the excavation could have stood for years based on the GSI of 80 but as a result of the orientation of the foliation around the boundary in the tunnel crown the rock quality is effectively reduced such that the stand-up time is only days to maximum a weak. Considering the natural scatter in stand-up time graphs, this means that little to no stand-up time should be anticipated in this rock. Of course, it should be pointed out that this effect is only encountered in the inner, shallow shell where low confinement promotes degradation. Furthermore, this degradation occurs right at the face and thus quickly stabilizes as the face advances.

Using the results discussed above, an order of magnitude shift in the SRF occurs as a result of the orientation of the structures. This effectively lowers the rock mass strength in areas close to the excavation boundary. This shift is shown in Figure 7 by the hatched boxes. This order of magnitude shift in the SRF chart represents the influence of structure around the circular tunnel.
and thus the Orientation Reduction Factor (ORF) required to identify areas that could be at a higher risk of stress induced failure in the inner shell (i.e. areas with low confinement levels of <0.5 MPa). This interpretation is consistent with the observed behaviour shown by Kaiser (2006). The approach presented here quantifies the influence of orientated weakness planes or structures around the boundary of a circular tunnel.

Figure 7: Shift in SRF required based on structural orientation around a circular tunnel due to structural anisotropy. Grey boxes, SRF ranges after Barton, (2002). Hatched boxes, SRF shift due to lowering of rock strength as a result of structural orientation (Bewick, 2008).

4.1 Case Example

Everitt & Lajtai (2004) found evidence at the Underground Research Lab (URL) of Atomic Energy of Canada Ltd. (AECL) that anisotropic rock conditions (i.e. structure, average grain size, variation in grain size, number and type of foliations, microcrack alignments and their orientations) may have dominated the excavation damage development. Foliation and other seemingly minor textural variations in the rock were shown to be major contributors to the observed variations in excavation damage, and to a lower than expected in situ rock strength.

Breakout notch development was found to be more extensive where the foliation was subject to regions of compressive stress concentrations tangent to the excavation surface. As discussed by Everitt & Lajtai (2004):

“…Subsequent extensional failure (buckling) and spalling along these surfaces was in some cases part of the initial stages of progressive failure. Where the layering was re-orientated or absent due to the presence of either fine grained dykes or some xenoliths, breakout development was much slower to develop and reduced in size.”

Since this is in part putting work in question that relates notch formation to stress alone (assuming isotropic rock strength) modelling was undertaken to assess this case example to assess weather Phase2 could reasonably predict the influence of the observed foliation on notch development.

As can be seen in Figure 8, the Phase2 model (see Bewick, 2008 for model parameters and in situ stress regime) appears to be able to predict spalling facilitated by joints and/or foliation.
Clearly the preliminary models need further refinement as numerous studies have shown that the notch can be matched by many models (Diederichs, 2003).

![Figure 8: URL example. a) & c) Phase² models with joint elements compared to corresponding field observations in b) & d) respectively. Figures 8b & d are reproduced from Évertt & Lajtai (2004). Material properties can be found in Bewick (2008).](image)

5 CONCLUSIONS

The influence of structural orientation was assessed for a 10 m diameter circular tunnel. The results compare well to observations in the field which suggest that:

- The location and extent of stress induced damage is not only a function of stress magnitude relative to intact rock strength and stress orientation but is also significantly affected by the orientation of structures and weakness planes such as systematic foliation near the excavation boundary.
- The presence of unfavorably oriented systematic structure around the boundary of an excavation can lead to stress induced breakouts to occur in areas that would not be predicted using current stress induced failure assessments.
- A span to spacing ratio of >10 is required for systematic structural layering to influence stress driven rock mass degradation processes.

Based on the parametric analyses, the influence of structural orientation around the boundary of the tunnel was determined to typically influence the rock within a 100° window, 50° on either side of a direction normal to the parallel structure orientation (i.e. β angles 0° to 50°). This data was then used to propose an Orientation Reduction Factor (ORF) similar to the Stress Reduction Factor (SRF) after Barton et al. (1974). The ORF concept was tested by Bewick (2008) on three case examples found in the literature (results not shown here) and found to produce reasonable results.
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REFERENCES


