

# When the Layer-Cake Bites Back: Tunnelling Challenges in Sedimentary Rock

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## ABSTRACT

Site Investigation, geomechanical design analysis, ground monitoring and support optimization are integral parts of the tunnel engineering process. The validity of the geotechnical approach to specific behaviour of the ground impacts constructability, excavation method selection, details of TBM design, support design, monitoring strategy and long-term performance of the project. This caveat is relevant for tunnelling within sedimentary rocks. Layering creates challenges for tunnelling if not considered during the design stage. Kinematics, structural and anisotropic response must be considered. Monitoring strategies must account for delamination or localized shearing. Support strategies, design and sequencing, must be compatible with this anisotropic response and the interlayer deformations. Geological formation designation is often based on consistent environments during the deposition phase. A deltaic formation, for example, may contain hard cemented sandstone, weak cross-bedded sequences as well as silty or clay-rich interbedding or lensing. Caution is required when interpreting geological descriptions for project sites. Site investigation, selection of analysis model and monitoring plan must link to ensure that the mechanics of the sedimentary rock are considered in excavation strategy, support design and monitoring (alert levels and response). Case examples will be discussed highlighting pitfalls and solutions (TBM, Mechanical and D&B) in sedimentary rocks.

## RÉSUMÉ

L'étude de site, l'analyse de conception géomécanique, la surveillance du sol, et l'optimisation de soutènement du massif rocheux font une partie fondamentale du processus d'ingénierie de tunnels. La validité de l'approche géotechnique à un certain comportement du sol affecte la constructibilité, la sélection de la méthode d'excavation, les détails de la conception du tunnelier, la conception du support, la stratégie de surveillance, et la performance à long terme du projet. Cet avertissement concerne spécifiquement les tunnels construits dans des roches sédimentaires. La stratification de ces roches crée des défis pour la création des tunnels si elle n'est pas prise en compte lors de la phase de conception. La réponse cinématique, structurelle et anisotrope doit être prise en compte. Les stratégies de surveillance doivent prendre en compte le délaminage ou le cisaillement localisé. La conception de soutènement et de séquençage doit être compatible avec cette réponse anisotrope et la déformation des couches intermédiaires. La désignation de la formation géologique est souvent basée sur les milieux de dépôt. Par exemple, une formation deltaïque, peut contenir du grès dur et cimenté, des faibles lits croisés, des lentilles argileuses ainsi qu'un entrelacement des couches riches en silt ou en argile. Par conséquent, la prudence est nécessaire lors de l'interprétation des descriptions géologiques pour les sites de projet. La caractérisation du site, la sélection du modèle d'analyse, et le plan de surveillance doivent être liés pour garantir que les mécaniques des roches sédimentaires sont prises en compte dans la stratégie d'excavation, la conception de l'appui et la surveillance (niveaux d'alerte et intervention). Des exemples de cas seront discutés en soulignant les risques et les solutions (TBM, Mechanical et D & B) dans les roches sédimentaires.

## 1 INTRODUCTION

Consideration of rockmass quality and integrity has long been a critical element of tunnel excavation method selection and initial support design (as well as final lining design). In general, the engineering behaviour of a zone of rockmass, through which a tunnel is to be built (or cavern constructed) is the result of intact rock strength, in combination with joint and fracture frequency, roughness, persistence, weathering, aperture, infilling and saturation condition (water flow and pressure). In addition, the in situ stresses play a significant role in rockmass behaviour (both in terms of low or relaxed stress conditions and high stress conditions).

Tunnel span, geometry and stand-up time (unsupported) play critical roles in the ultimate stability and support demand for an underground excavation. Figure 1 represents the components that influence the engineering behaviour of a tunnel-scale rockmass.

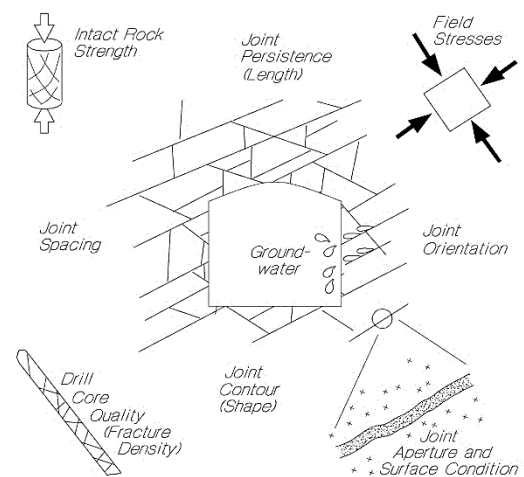


Figure 1: Engineering components of a rockmass (Hutchinson and Diederichs 1996).

The conversion of this complex array of components into a workable index for system performance was accomplished through the use of rockmass classification systems. These systems included some or all of the components in Figure 1. The most popular and long-surviving systems used in tunnelling were and are RQD, RMR and Q. These systems were typically calibrated to varying degrees, using case histories, such that they could be used as predictors for support requirements, maximum span and stand-up time.

RQD (Deere 1966) was an indirect measure (0-100) of joint spacing or frequency with some influence of persistence. This empirical system predates the Apollo space program, the first commercial semiconductor RAM device, and was in use when the Toronto Maple Leafs last won a Stanley Cup. In spite of its age and simplicity, it still sees routine application today. It is computed as the sum of the length of all pieces of intact core (or intact segments along a ruler placed against a wall) greater than 10cm, taken as a percentage of the total length examined.

The Q system (Barton et al 1974) starts with RQD as a base to consider fracture frequency and then multiplies and divides by factors that account for the number of oriented joint families or sets (Jn), the joint roughness (Jr), the Joint alteration (Ja), the degree of moisture, water flow or pressure (Jw) and the in situ stresses (versus intact strength) or discrete weakness features (SRF):

$$Q = RQD / J_w \times J_r / J_a \times J_w / SRF \quad [1]$$

The Q-system predates the release of the first personal computer, the first Rubik's Cube, the filming of the original Starwars epic, and the divorce between Cher and Sonny Bono. Q is still used in mining and tunnelling worldwide.

The RMR system (Bieniawski 1976, 1989) is an index (0-100) that is the simple sum of 5 ratings that consider the intact rock strength (A1), RQD and fracture spacing (A2 and A3), joint roughness, alteration and infilling (A4) and water flow and pressure (A5). This is the only system to also consider the orientation (with respect to the tunnel) of major continuous planes of weakness, including bedding, through the adjustment factor (B):

$$RMR = A_1 + A_2 + A_3 + A_4 + A_5 + B \quad [2]$$

RMR predates the first computer and operating system by Bill Gates, the expensive spectacle of the Montreal Olympics, the first Concorde flight and the opening of the CN Tower in Toronto. RMR remains a staple of Geotechnical Baselines, and Observational Design recording for tunnels, to this day around the world.

Two of the most prolific design charts still in use today based on these classification systems are shown in Figure 2. These charts do not require any additional analysis in order to go from characterization to final design. This was appropriate and a critical engineering advance at the time the systems were developed. Other charts based on RQD and other classification systems have fallen out of favour. The Q system chart shown as been updated over the years (Grimstad et al 1993, NGI 2015) and the RMR system has also seen an update (Bieniawski 1989) affecting the calculation of the component factors.

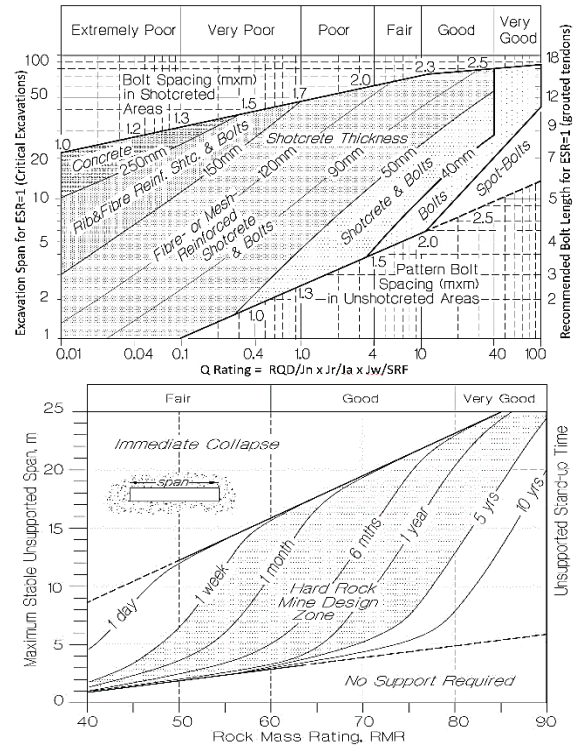


Figure 2: (top) Q Support after Grimstad et al. 1993; (bottom) RMR Stand-Up Time (based on Bieniawski 1989). Both plots from Hutchinson and Diederichs (1996).

The GSI system (currently as per Hoek et al 2002) is not a standalone classification system, but was designed to account for fracture intensity, number of joint sets and block-size, as well as joint roughness, alteration and infilling. A simplified version of the GSI assessment chart is shown in Figure 3.

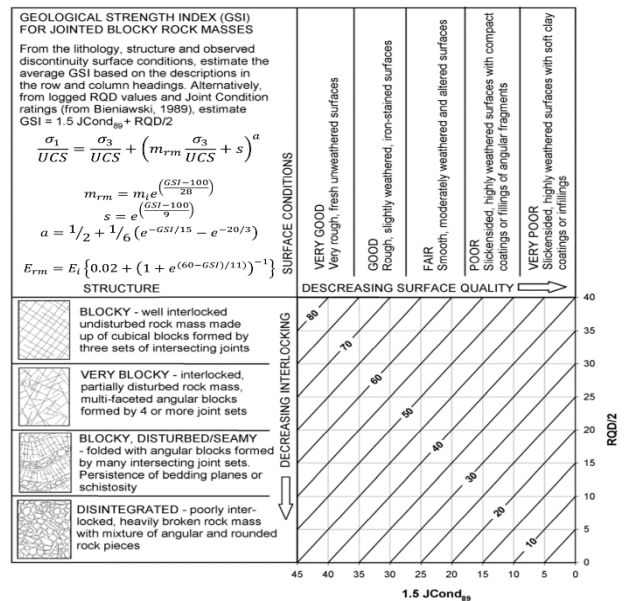


Figure 3: GSI system for rockmass strength and stiffness (after Hoek et al 2013) - (JCond89 = RMR Factor A4).

The GSI system, conceived in the year Mt. St. Helens erupted (1980) and refined into its current graphical form during the collapse of the Soviet Union, is designed to provide an index that can then be used to factor the intact rock strength and stiffness parameters, giving "equivalent continuum material" rockmass parameters (based on the Hoek-Brown relationship for rock strength fully described in Hoek et al 2002) for use in analytical solutions for material behaviour or for continuum numerical analyses of tunnel response. A comparison of yield and displacement of a tunnel, based on intact rock strength and stiffness, and on the strength and stiffness predicted by the GSI system, is shown in Figure 4 using a commonly available finite element software. The use of GSI today, in tunnelling projects, is regional but widespread around the globe for analyses such as these.

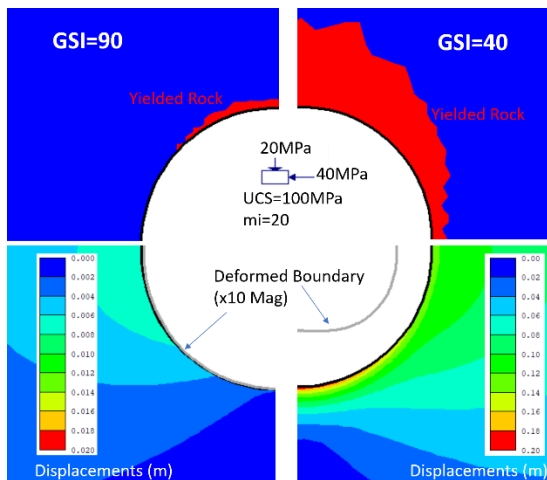


Figure 4: Simple continuum FEM analysis illustrating effect of isotropic GSI on rockmass (circular tunnel in rock).

From 1966, computers available for engineering analysis have evolved from memory chips offering 8k of RAM and permanent storage on floppy disks, to the ever evolving and powerful systems we use today. The state-of-the-art in engineering analysis has progressed, in this time, from the solution of closed form solutions for tunnel stress around a circular opening, to inelastic analysis (finite element and finite difference) of continuum material behaviour, to user-friendly software capable of analyzing discrete structural entities such as joints, bedding and propagating fractures (discrete element codes and hybrids).

All of these systems, RQD, Q, RMR and GSI consider the rockmass to be an equivalent isotropic continuum, weaker and softer than the intact rock component alone. While RMR does consider the impact of weakness planes and Q does have a factor to adjust for discrete structure, the final index in all cases is specified without mechanistic consideration of discrete and systematic weakness planes such as those found in sedimentary, volcanoclastic and metamorphic rockmasses.

In both high stress and low stress environments, failure to consider the mechanical impact, of these persistent weakness planes, on the tunnel stability and support strategy can and often does result in major deviations from anticipated tunnel response including collapse.

Given the power of modern computers, and the ability of commercially available (and user-friendly) software to model discrete features within the rockmass, there is no reason to forego consideration of laminated ground, bedding and dominant anisotropy at the preliminary or final design process.

In this paper, a number of examples will be described to illustrate the impact of such an omission, either at the bid stage (lack of consideration by owner's or contractor's engineer) or during final design and construction.

## 2 MECHANICS OF LAMINATED GROUND

The impact of laminated strata or bedding on tunnel stability is dependent on the relative spacing of the weakness planes (relative to the tunnel span), the heterogeneity of strength and stiffness between the various layers, the cross structure (vertical joints cutting horizontal bedding, for example) and the in situ stress (unconfined, confined or overstresses). Tunnel boundary geometry can exacerbate or mitigate some of these impacts.

### 2.1 Geometric Mismatch

One of the most common challenges in TBM (tunnel boring machine) excavation is literally akin to the age-old metaphor of the square peg and the round hole. Bedding planes that are undercut by a tunnel must flex and generate internal load transfer to the abutments in order to offset the destabilizing effects of gravity. When a circular TBM driven profile (or an arched roof in conventional tunnelling) intersects the strata, it will create unsupported haunches that are inherently unstable for thinly bedded strata as shown in Figure 5a. These example results are generated with a FEM model with simple Goodman Joint elements (Goodman 1980). Discrete element analysis would yield similar results. In addition, if the bedding or lamination is slightly inclined, a stable horizontal roof slab spanning abutments cannot be achieved (Figure 5b). In these example cases, simple isotropic equivalent rockmass analysis using GSI would show no failure.

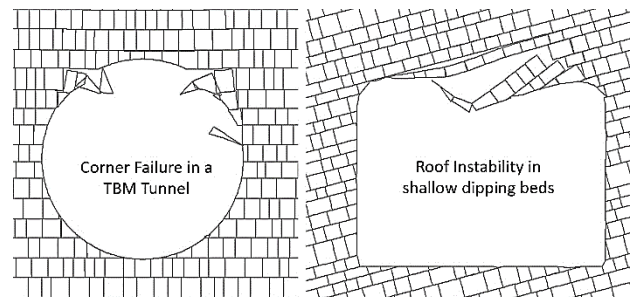


Figure 5: Examples of geometrically influenced structurally controlled failure in bedded rockmasses

As the bedding plane represents both a shear weakness surface but also a plane of near-zero perpendicular tensile strength, block or slab fallout is inevitable if not immediately supported with stiff passive (grouted) or tensioned active bolts at the face (a challenge in TBM excavation due to the length of the shield around the cutter head). The bolting must encompass the

haunches as well as the central roof area. Figure 6 illustrates to TBM cases where support design and sequencing were inadequate resulting in conditions such as those in Figure 5. In a TBM drive such as these, discrete structural analysis for rock that is otherwise of good quality (medium to high GSI) would indicate the instability potential and lead to design features like adjustable roof shield (to apply initial support pressure) and immediate bolting.

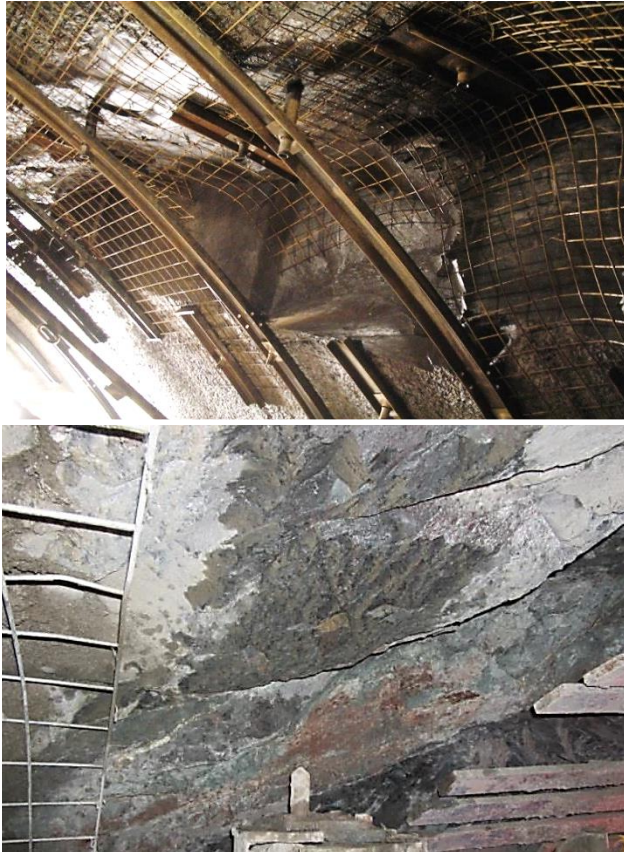


Figure 6: Examples of (top) corner failure in a TBM tunnel and (bottom) roof instability an overbreak (also TBM in this case) due to shallow dipping bedding structures.

## 2.2 Stability of Laterally Constrained Jointed Strata

Horizontally laminated ground can be stable above a flat roof provided that the layers have sufficient thickness and stiffness. Stability is achieved when the roof slabs deflect slightly, generating internal stresses parallel to the beam.

In an ideal setting with only intact slabs, the stability can be assessed using classic beam theory with various assumptions for the fixity of the abutments ("roller", "pin", "fixed"). For this ideal case, Figure 7a illustrates the basic premise that, considering a half span, the moment created by the beam self-weight about the abutment is balanced by the moment generated about the slab neutral axis by the integrated stresses in the beam (compression at the top of the midspan, varying linearly through zero in the middle and tension on the underside of the slab). Most laminated rock layers, however, would inevitably have some cracks or joints through the slab, rendering the tensile strength of the rockmass parallel to the beam effectively zero. Beam

theory does not apply in this case as tension cannot balance the compression in the beam.

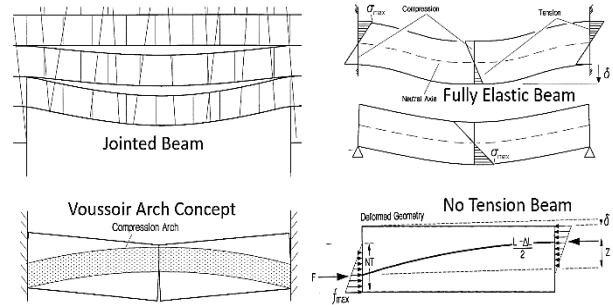


Figure 7: (Clockwise from top left) Concept of a bedded roof composed of vertically jointed beam; stresses during deflection of an elastic beam; compression arch generated in a half span with no tension; complete Voussoir arch in situ. (after Diederichs and Kaiser 1999a)

A technique known as Voussoir analysis can be used to assess the stability in this case (Evans 1941, Beer and Meek 1981). The assumption is that as the beam deflects, compression is generated within a portion of the beam from the lower abutment contact through the top of the midspan. The lower part of the midspan is stress free as tension cannot be generated.

The deflection in this case is more than that for a simple intact beam. The separation between the resultant compression force vector at midspan and the force vector at the abutment creates the balancing moment to achieve stability. Failure occurs when the slab cannot generate enough internal compression before the deflection reaches a critical value and the beam "snaps-thru". Failure can also occur through crushing of the beam (excess deflection and weak rock) or through sliding at the abutments (thicker slab with weathered vertical joints and relaxed abutment conditions)

This technique was refined for practical application by Diederichs and Kaiser (1999a). A simplified span-stability chart for an unsupported roof is presented in Figure 8.

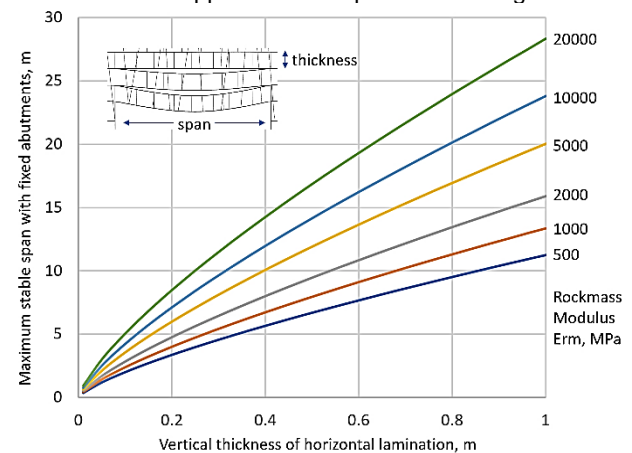


Figure 8: Max. span vs lamination thickness and stiffness

Note that the rockmass stiffness ( $E_m$ ) in the chart above can be determined, where the normal stiffness ( $K_n$ ) of the joints can be tested, by the relationship:

$$\frac{1}{E_{rm}} = \frac{1}{E_i} + \frac{1}{K_n S} \quad [3]$$

where  $E_i$  is the intact stiffness and  $S$  is the spacing of the vertical cross-joints within the bedding layers.

While the Voussoir concept has been used with success in major projects such as the Sydney Opera House parking garage (Pells et al 1994) in competent bedded sandstone (where the conditions and response are well represented by Figure 8), there are a number of geological factors than can lead to tunnel and cavern failure during construction if not appreciated in advance. These are addressed in the models of Diederichs and Kaiser 1999a and 1999b and include:

- A thinning of the bedding layers during advance leading to a critical limit with respect to span (Figure 8).
- Beam softening over time or due to a change in alteration/weathering condition (reduction of  $K_n$ ) of the sub-vertical cross joints (Equation 3 and Figure 8)
- A competent layer overlain by thin beds or weaker rock creating a surcharge condition on the main roof slab leading to delayed failure if under-supported. (Figure 9a)
- Abutment relaxation due to subsequent adjacent tunnelling or mining in the area. (Figure 9b). An equivalent condition can occur if filling or sidewall material is lost through the effects of seepage.

In all cases, immediate support in the form of short to medium length bolts is required (complete coverage across roof. Abutment bolts should have some shear resistance (grouted bolts) to limit interbed slip. The goal here is to create a thicker beam to limit deflection and gain stability.

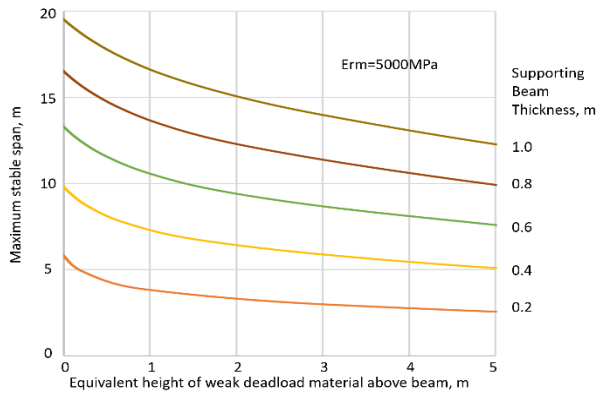


Figure 9a: Effect of weaker layers above beam

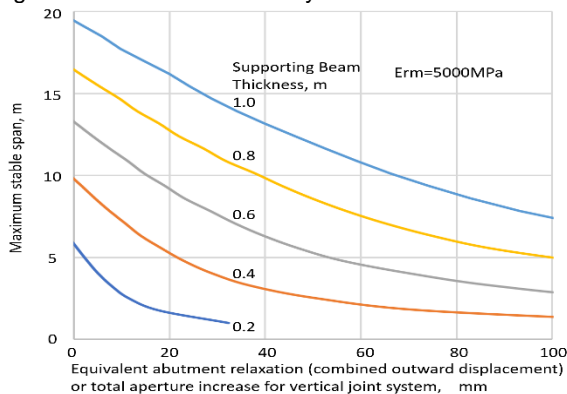


Figure 9b: Effect of relaxation or material loss

Stiff grouted bolts that are too long and of uniform length, installed at the face, may create a beam with insufficient deflection, limiting the flex-induced confining stress on the abutments and creating slab failure through abutment slip. Long bolts can be delayed one round in this case.

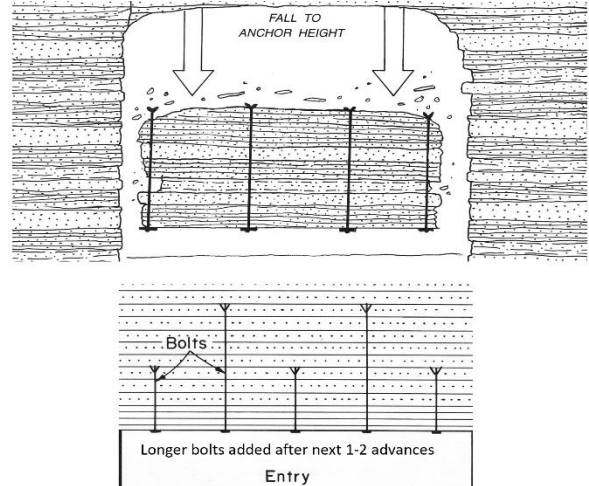


Figure 10: Effect of dense, stiff bolting (modified after Moebs and Ellenberger, 1982).

Figure 11-15 illustrate a number of failure cases where a jointed beam structure has failed due one or more of the reasons given above.



Figure 11: Roof instability due to thinning beds



Figure 12: Roof collapse due to time-dependent relaxation



Figure 13: Collapse and chimney failure due partially to upward weakening beds and Voussoir beam surcharge



Figure 14: Roof collapse above subway construction due to weak rock/soil surcharge above thin roof beam



Figure 15: Progressive collapse of roof slabs due to weathering/seepage loss in vertical joints and abutment relaxation (after Diederichs and Hutchinson 2019).

Laminated rock originating from a sedimentary sand environment such as the Hawkesbury sandstone in Sydney, Australia can often have primary structures that are not horizontal or vertical. Cross bedding is a sedimentary structure formed in deltaic environments. Figure 16 shows the weak cross bedding in certain layers of the otherwise competent sandstone in Sydney. These inclined cross layers rob the sandstone of beam forming capacity and are isolated layers that can appear unexpectedly in a tunnel without proper vertical probing.



Figure 16: Crossbedding in layered sandstone

To illustrate the importance of analyzing the bedded layers as discrete features (compared to an equivalent rockmass continuum), Figure 17 compares a continuum finite element model of a proposed subway station in Sydney (unsupported in this case), with a model containing discrete bedding planes (joint elements as per Goodman 1980). The impact of strata and interbed mobility is clear and has implications for support design and sequencing.

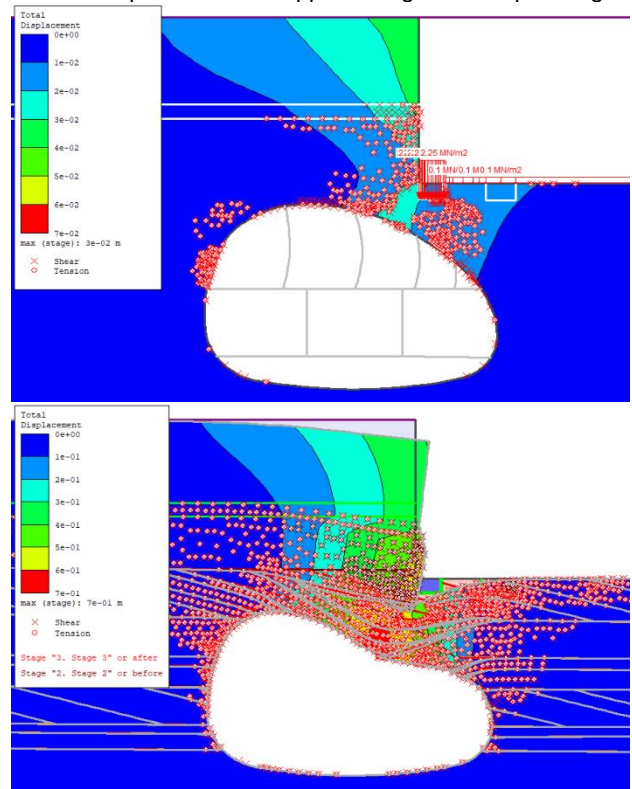


Figure 17: Comparison between continuum analysis (based on GSI) and discrete analysis of structure. Note the difference in displacement scales (plotted deformation x 5).

In the figures above, the impact of cross bedding within two of the roof layers is clear. In Figure 18, the impact of the presence of discrete bedding and the influence and orientation of the crossbedding with respect to the cavern and adjacent building foundation (footing loads are modelled but not shown). Moments in the final liner are shown for comparison (support includes bolting as shown).

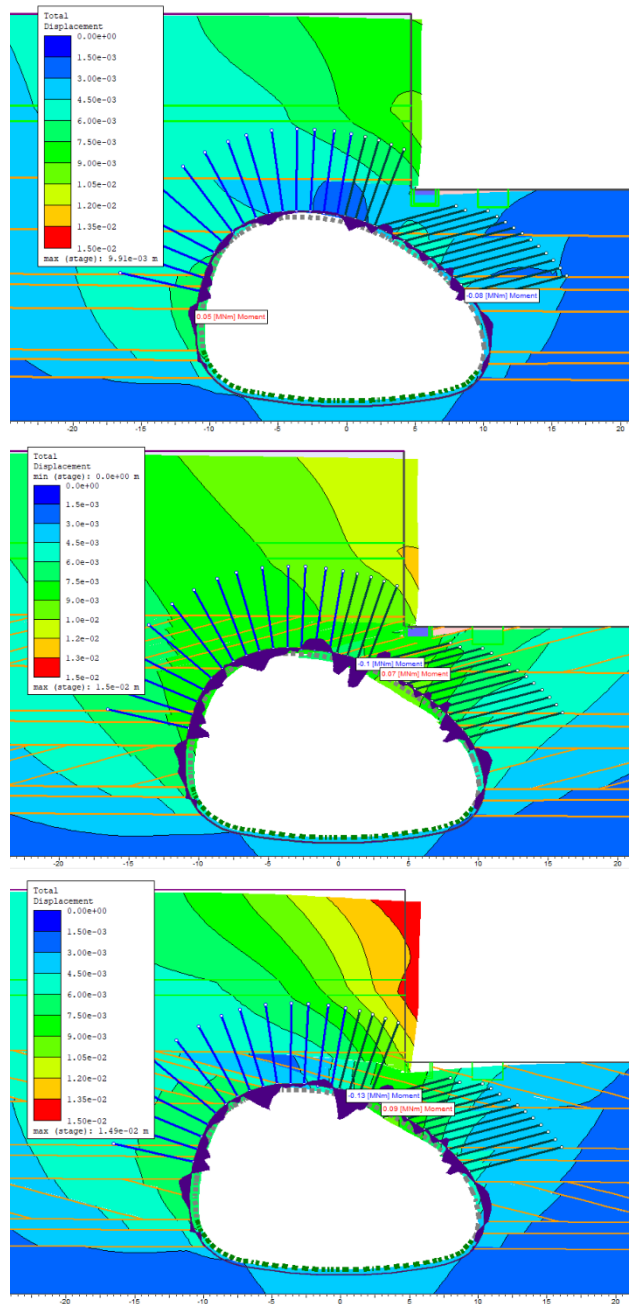


Figure 18; Comparative analysis of the cavern in Fig. 17 with full liner and bolt support. The effect of no cross-bedding (top) and two alternate dip directions of cross-bedding (middle & bottom) is shown. In this example, liner moments are plotted as are total displacements during cavern excavation (plotted deformation x 100).

Detailed pre-construction drilling along the tunnel alignment is essential for design-build projects in sedimentary rock where such heterogeneities in layering or interlayer structure are expected. In the case of projects employing observational design, it is critical not to evaluate only the tunnel face and immediate boundary in each round, but to also utilize engineering geology skills to project layers forward and to incorporate vertical probe holes (endoscope monitoring) in the roof at regular intervals to anticipate conditions above the immediate roof layers. These holes should be at least 1/3 to 1/2 of the span in length. Conventional look-ahead horizontal probe holes are less effective in this environment for identifying subhorizontal structures and hazards.

### 2.3 Stability of Overstressed Strata

Horizontal strata can pose a very different challenge when high horizontal stresses are present. Where brittle damage and fracture potential is present, the presence of laminar planes of weakness above a tunnel roof can create the potential for stress channelling (leading to increased stresses immediately above the roof) due to interlayer slip (as shown in Figure 19). In extreme cases, interlayer slip can result in the shearing of bolts with and without grout (Figure 20). Fracture propagation along incipient planes of weakness, exacerbating damage and failure in the roof (Figure 21).



Figure 19: Slabbing in the competent sandstone layers due to high horizontal stresses in eastern Sydney (after Oliveira and Diederichs 2017).

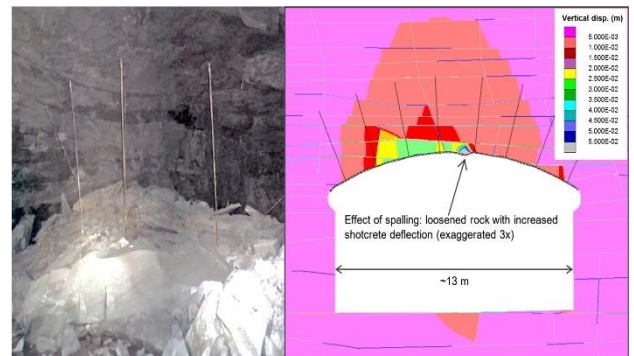


Figure 20: Slab failure due to stress induced joint slip and shearing of bolts, prior to grouting, in sandstone (after Oliveira and Diederichs 2017) above the unstable slab..

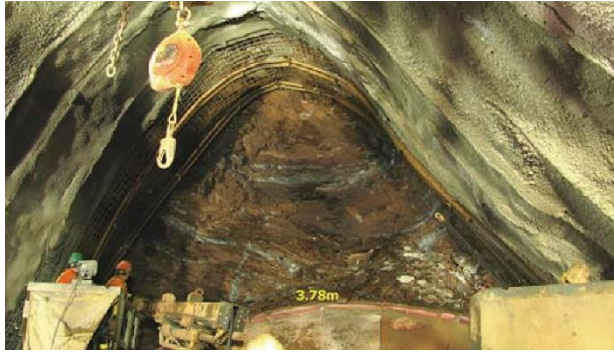


Figure 21: High overbreak (up to 4m) in 14m diameter Niagara-Beck TBM tunnel, in mudstone/shale with high horizontal stresses (Perras et al. 2014)

In very highly stressed environments, laminated ground can present a severe buckling or shearing hazard with unpredictable dynamic release. This can lead to severe and dangerous rockbursting in tunnels and mine openings as shown in Figure 22.

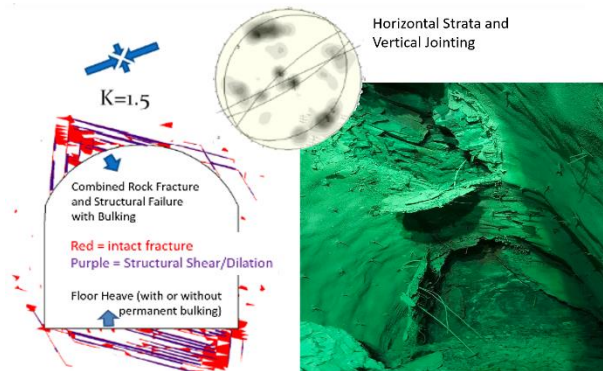


Figure 22: Rockbursting controlled by heterogeneous laminations of volcanics, pyroclastics and sedimentary lenses in conditions of high horizontal stress.

### 3 CONCLUSIONS

As illustrated in many of the examples above, the interaction of stress with sedimentary or other discrete layering (as well as accompanying or conjugate cross structure) can only be analyzed with modern discontinuum tools. For lower stress environments, the impact of layering must be taken into account using a voussoir approach or discrete structural analysis. The difference in behaviour is clear between the continuum approach (isotropic rockmass strength and stiffness based on GSI) and the discrete approach incorporating the planes of weakness. The common rockmass classification tools do not allow for full consideration of the structural control due to sedimentary layering. Sophisticated numerical tools are readily available for the trained tunnel engineer. It is no longer acceptable to rely solely on rockmass classification or continuum analysis for tunnel design or tunnel construction and support management in these conditions. Probing ahead of the face in these conditions is often less effective than vertical drilling and longitudinal forward projection of structure. In most conditions, immediate shorter stiff support is needed to maintain thicker layering followed by longer tendon support and structural liners if needed.

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