Rock Strength Anisotropy and Its Importance in Underground Geotechnical Design

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ABSTRACT

Although rock strength anisotropy is a well-known phenomenon in rock mechanics, its impact on geotechnical design is often ignored. No widely accepted guideline currently exists for its characterisation. Anisotropy can significantly control the failure mechanism and progression of damage into the rock mass, particularly in high-stress conditions. As such, it can play a key role in the stability of underground excavations and subsequent geotechnical design.

This paper introduces the concept of anisotropy at both intact and rock mass scales. It then presents guidelines for measuring anisotropy and assigning rock mass strength properties. An improved unified constitutive model (IUCM), which can account for more complex failure mechanisms, is then briefly described. This IUCM is used to better understand the typical responses of anisotropic rocks to underground mining and to develop an improved damage classification scheme for these conditions.

The application of conventional design methods in anisotropic conditions is shown to lead to significant errors. Two case histories are then presented, illustrating how the proposed methodology can improve the reliability and accuracy of the geotechnical analysis when compared with conventional design methods.

INTRODUCTION

The significance of rock strength anisotropy (anisotropy) in geotechnical design is often ignored or underestimated. This is partly because most geotechnical design methods (whether empirical, numerical or analytical) are largely developed for isotropic and not anisotropic rock mass conditions. Thus there is a tendency to ignore its impact to simplify the design process or apply conventional design methods.

However, anisotropy can play a key role in the stability of underground excavations and subsequent geotechnical design. Experience suggests that in many cases, anisotropy can even override other geotechnical factors such as stress in controlling the failure mechanism. In high-stress conditions, anisotropy can significantly change the time dependent failure mechanism and progression of damage into the rock mass.

Recent literature, such as that of Sandy *et al* (2010); Vakili, Sandy, and Albrecht (2012); Hadjigeorgiou *et al* (2013); and Vakili *et al* (2013) outlines the typical response of anisotropic rock masses to increasing stress levels and shows examples of geotechnical design in these conditions.

This paper first defines the concept of rock strength anisotropy. It then provides brief guidelines for data collection and selection of input parameters for geotechnical analysis considering this condition. A recently developed and improved IUCM is described. This is applied in sensitivity analyses and two case studies to better understand the typical responses of anisotropic rocks to underground mining. It also highlights how suitable geotechnical analysis can improve the reliability of design in anisotropic rock mass conditions.

DEFINITION OF STRENGTH ANISOTROPY IN ROCK MATERIAL

Anisotropy is a mechanical property of rock that makes the rock's strength (or other material properties) directionally dependent. Unlike isotropic rocks, which have similar strength properties in all directions, anisotropic rocks can have significantly lower strength when loaded along their weak orientation.

Anisotropy is usually caused by some obvious 'fabric' in the rock material, such as schistosity, foliation or bedding. The intensity of anisotropic behaviour (in other words, the difference in directional strength) is often controlled by the amount and characteristics of flaky and elongated minerals present (such as mica, chlorite or amphiboles). This influence tends to be significant even at the scale of a laboratory test specimen (Palmström 1994; Brady and Brown, 2005). Two forms of anisotropy are often present in rock material – intact rock anisotropy and rock mass anisotropy.

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Intact rock anisotropy is a result of natural fabrics, such as schistosity, foliation and bedding, constituting the rock. This can cause directional dependency even in a homogeneous intact anisotropic rock at very small scales.

Rock mass anisotropy, on the other hand, is large in scale and is often due to the presence of well-defined and persistent joint sets in the rock mass. In the majority of cases, rock mass anisotropy occurs when the constituting rock types exhibit intact rock anisotropic behaviour; however, it is also possible for a rock mass to exhibit directional-dependent behaviour without any small-scale (intact) anisotropy. This is often the result of various *in situ* stress histories or other geological occurrences that have caused well-defined joint sets in the rock mass.

This paper focuses on rock types that have the 'intact rock anisotropy' characteristic and therefore exhibit directional dependency at both small and large scales. Typical rock type examples are gneiss, mylonite, migmatite, quartz schist, mica schist, hornblende schist, slate, shales, phyllite and coal.

As shown in Figure 1, the compressive strength of anisotropic rocks can vary significantly with respect to plane of weakness depending on the loading direction. As shown by Tsidzi (1986, 1987a, 1987b), Ramamurthy, Venkatappa Rao and Singh (1993) and Hoek and Brown (1980), the lowest strength value ($\sigma_{c\ min}$) occurs when the orientation of the anisotropic fabric element (bedding, foliation) to the specimen loading axis (β angle) is between 30° and 45°. The highest strength ($\sigma_{c\ max}$) is achieved when the orientation is either 0° or 90°.

The ratio of $\sigma_{c\ max}$ over $\sigma_{c\ min}$ provides an indication of anisotropy intensity for various rock types. This ratio is often referred to as the *anisotropy factor* or *anisotropy ratio*.

Table 1 shows a classification of anisotropy and associated factors, extracted from other literatures (Tsidzi 1986, 1987a, 1987b; Singh, Ramamurthy, Venkatappa Raol 1989; Ramamurthy, Venkatappa Rao and Singh, 1993; Palmström, 1994).

GUIDELINES FOR DATA COLLECTION AND SELECTION OF INPUT PARAMETERS

The potential for anisotropic behaviour is often overlooked in data collection or in laboratory tests on rock types that commonly exhibit such behaviour.

TABLE 1

Classification of anisotropy intensity in various rocks (modified from Tsidzi, 1986, 1987a, 1987b; Singh, Ramamurthy and Venkatappa Rao, 1989; Ramamurthy, Venkatappa Rao and Singh, 1993; and Palmström, 1994).

Anisotropy classification	Example rock types	Anisotropy factor
Isotropic	Quartzite, hornfels, granulite	1–1.1
Low anisotropy	Quartzofeltspatic gneiss, mylonite, migmatite, shales	1.1–2.0
Medium anisotropy	Schistose gneiss, quartz schist	2.0–4.0
High anisotropy	Mica schist, hornblende schist	4.0–6.0
Very high anisotropy	Slate, phyllite	>6.0

To ensure adequate understanding of anisotropic behaviour, the following guidelines should be followed when selecting and preparing core samples for laboratory testing:

- When selecting samples for uniaxial compressive strength (UCS) and triaxial testing, preference should be given to samples with β angles from 30° to 45°, close to 0°, or close to 90°. Ideally, the total number of samples collected should include an equal number from each of the three angle categories.
- When selecting samples for Brazilian or tensile strength testing, preference should be given to samples with β angles close to 0° and 90°.
- Prior to any testing, the angle between the weakness plane (bedding, foliation) and the specimen-loading axis (β angle) should be recorded for each sample. For Brazilian testing, these angles are measured in a plane that is perpendicular to the core axis. Unlike other tests, the β angle can be variable depending on the position of the samples inside the loading platens.

To enable maximum understanding of anisotropic behaviour of tested samples, the following guidelines should be followed when conducting laboratory tests and interpreting the results:

- It should be noted that the majority of UCS tests on anisotropic samples fail in shear (as shown in Figure 2). This is different to UCS results for isotropic rocks, where samples often fail in axial-splitting mode.

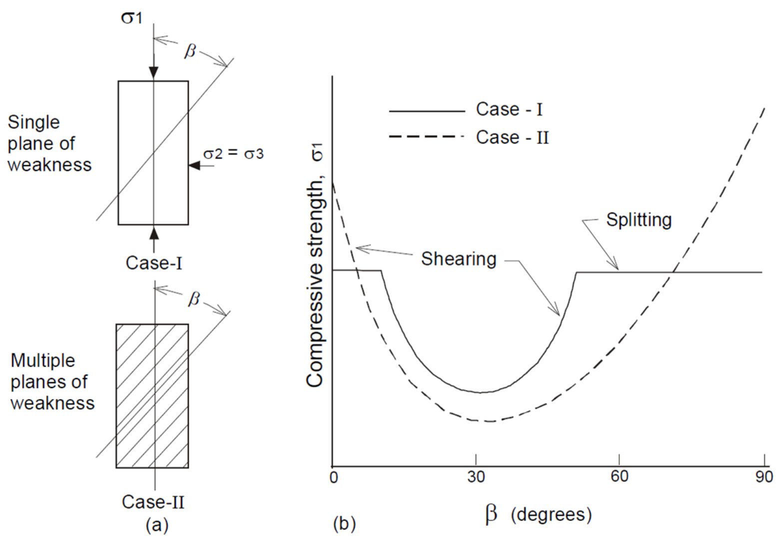

FIG 1 – Theoretical variation of uniaxial compressive strength depending on orientation with respect to anisotropy plane (after Ramamurthy, Venkatappa Rao and Singh, 1993, as referenced by Palmström, 1994).

- UCS testing should also be completed for samples with a β angle between 30° to 45° and for samples with a β angle close to 0° or 90°. To relate the UCS values to β angles, UCS results should be plotted similar to the way shown in Figure 2.
- For each set of β angles tested, the Triaxial tests should be conducted over a range of confinement stress (as shown in Figure 2).
- Tensile strength testing, whether Brazilian or direct pull testing, should also focus on capturing the directional dependency of tensile strength.
- Special care should be taken when conducting Brazilian tests on core sample with the anisotropy plane parallel to the core axis. In these cases, disks should be rotated inside the test rigs for different tests, to capture the variability of results due to anisotropy.

To estimate the parameters for Hoek–Brown criterion, it is best to use all data from UCS, triaxial and tensile strength testing and include them for curve-fitting purposes. For this purpose, the following parameters need to be derived from testing results:

- elastic modulus of the intact rock
- $\sigma_{c\,max}$ and $\sigma_{c\,min}$
- anisotropy factor ($\sigma_{c\,max}/\sigma_{c\,min}$)
- Hoek–Brown constant mi for tests conducted on samples with β angle between 30° to 45° ($m_{i\,min}$)
- Hoek–Brown constant mi for tests conducted on samples with β angle close to 0° or 90° ($m_{i\,max}$).

These parameters, together with the geological strength index (GSI) of the rock mass, can then be used to establish the highest and lowest rock mass Hoek–Brown failure envelopes. This can be used for further analysis and application in the numerical models.

THE IMPROVED UNIFIED CONSTITUTIVE MODEL (IUCM)

As mentioned, anisotropy is one of the most important factors to consider in geotechnical analysis of anisotropic rocks. If ignored, it can lead to significant errors and design problems. It can become even more significant when other complex factors are present in a mining scenario, such as elevated or highly deviatoric stresses.

Two factors often control the quality and reliability of the results of any geotechnical analysis – suitability of the analysis tool and the failure criterion (constitutive model) used.

It is important to note that in mining conditions with moderate to significant complexity levels, conventional analysis tools and constitutive models (failure criteria) are not able to represent the complex mechanisms involved. As such, in more complex conditions more advanced analysis tools and failure criteria need to be applied.

Figure 3 shows a qualitative matrix that can be used to estimate the level of complexity involved with a particular mining project. Table 2 summarises the applicable analysis tools associated with each complexity level.

In the case of anisotropic rock, the analysis tool and selected failure criterion should be able to explicitly model both intact rock failure and anisotropy plane failure. The significance of these factors is demonstrated in the case studies presented in this paper; however, conventional methods of analysis assume an isotropic material response

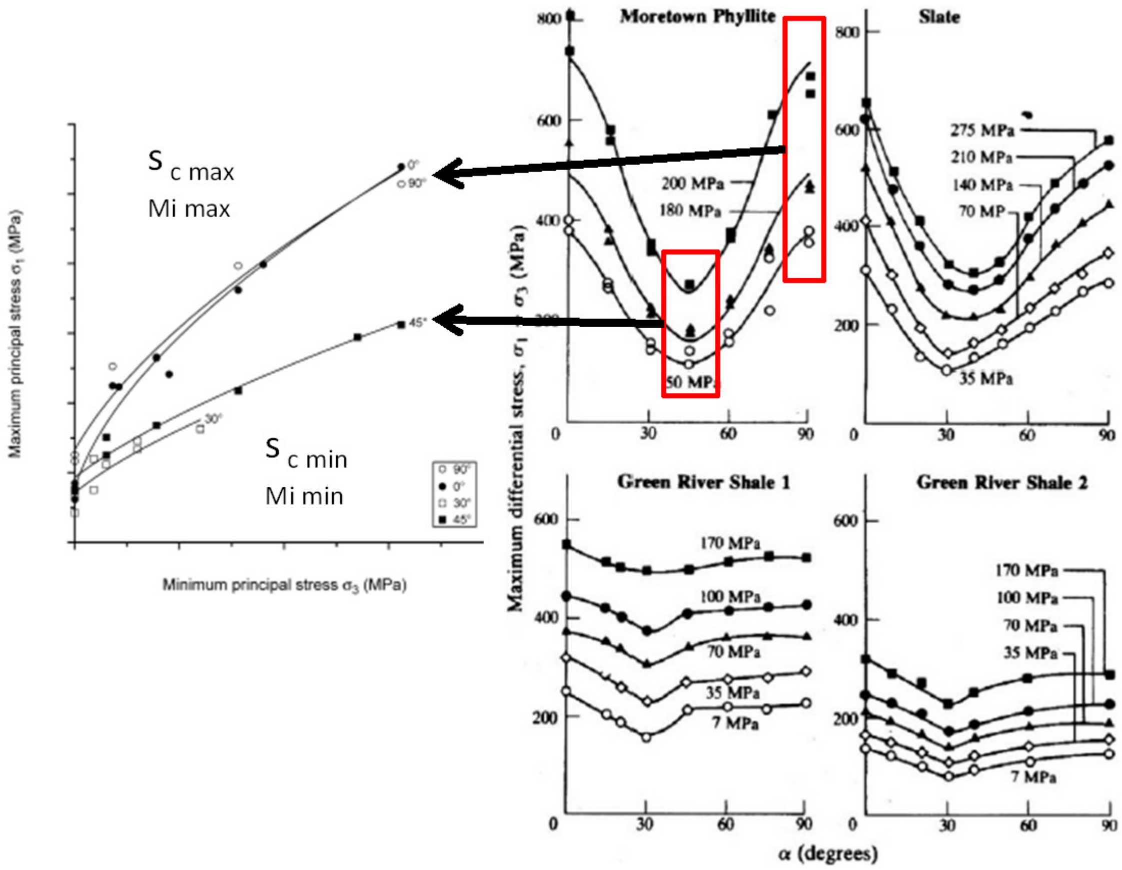

FIG 2 – Theoretical variation of triaxial testing results depending on orientation with respect to anisotropy plane and subsequent minimum and maximum Hoek–Brown failure envelopes (after Donath, 1972; McLamore and Gray, 1967; Brady and Brown, 2005; Saroglou and Tsiambaos, 2008).

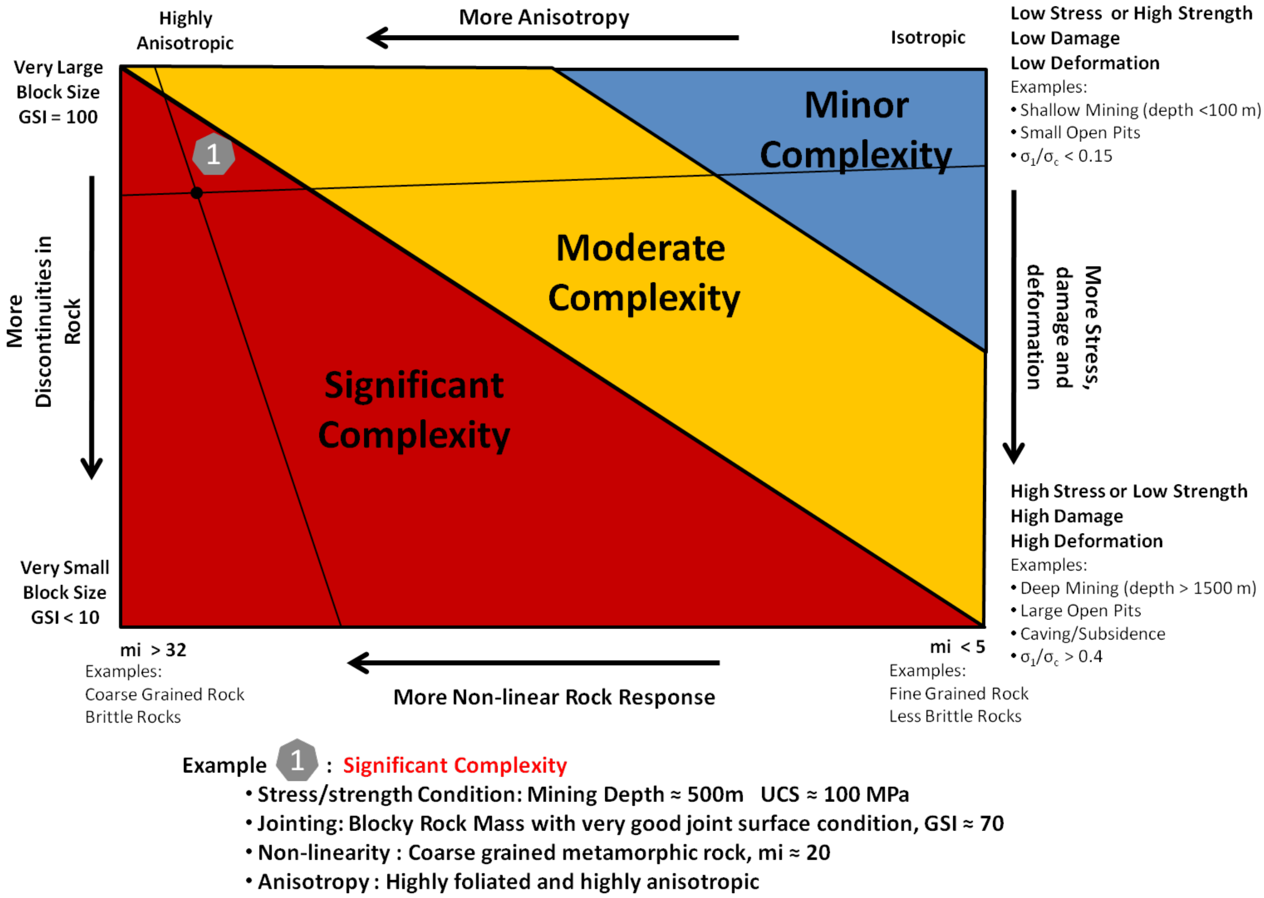

FIG 3 – A matrix for evaluation of degree of complexity involved in various geotechnical conditions (see example for clarification on how to use).

TABLE 2

Guideline for selection of appropriate analytical method for each complexity level introduced in Figure 2.

Reliability class	Applicable analytical methods
Minor complexity	Most conventional analytical and empirical techniques can be applied. Empirical methods should ideally be used only at planning stage and analytical methods for more detailed design. Simple failure criteria such as linear Mohr–Coulomb can be sufficient to forecast the behaviour. Conventional analytical and empirical methods are usually the most efficient methods for this class.
Moderate complexity	Empirical methods should be applied with great caution considering their likely limitations. Elastic and implicit inelastic modelling methods should also be applied with caution and only when calibration data exists. The above methods should only be used for planning purposes if all their limitations are fully understood. Simple failure criteria should be applied with great caution and only if the problem area is very localized and limited depth (stress) variation is expected. Explicit inelastic models incorporating more advanced failure criteria provide more reliable results. If no calibration data exist, application of elastic or implicit inelastic codes is not recommended. In this case, a combination of empirical and explicit inelastic methods is recommended. The most efficient and reliable design for this class would usually result from a combined application of advanced analytical methods (such as explicit inelastic) with conventional methods (such as empirical systems).
Significant complexity	Application of empirical systems is generally not recommended unless there is sufficient evidence that exactly the same mining conditions were originally accounted for during the development of the empirical system. Application of elastic and implicit inelastic models is generally not recommended unless significant calibration data exists for exactly the same condition under study, both in terms of stress and geometry. An explicit inelastic model, which incorporates advanced failure criterion, is currently the only reliable tool that can be applied for this class of problems.

and therefore do not account for these two complex failure mechanisms.

An IUCM was developed as a result of previous works on rock damage processes and review of previous literature. After being tested on a number of well-documented case histories, it was shown that this unified model could forecast the extent and severity of damage more accurately than other conventional methods.

A detailed discussion on IUCM and its theoretical basis will be included in future publications and is outside the scope of this paper; nevertheless, here are the key components and features of this model:

- For the peak failure envelope of the rock matrix, IUCM uses a Hoek–Brown (1980) failure criterion to determine the instantaneous Mohr–Coulomb parameters (C and Phi) at each level of confining stress. These instantaneous parameters are updated in real time as the model runs and as new phases of confinement are formed due to nearby damage or geometrical changes.

- For the residual state of the rock matrix, IUCM assigns a linear Mohr–Coulomb envelope. Properties of completely broken and crushed rock are applied for the residual state of the material using a cohesion and tensile strength of zero and a friction angle of 45° (as recommended by Lorig and Varona, 2013). At low confinement levels, the linear nature of the residual envelope replicates cohesion and friction softening. At high confinement levels, it replicates cohesion softening and friction hardening (see Figure 4). This feature of the model allows progressive failure to occur near the boundary of the excavation. At the same time, it limits the propagation of yield or plasticity zones away from the excavation boundaries (as observed in the field).
- The critical strain (see Figure 4) is chosen based on equations suggested by Lorig and Varona (2013). In this method, critical strain values are determined based on model zone size and the GSI values. This critical strain can also be adjusted when synthetic rock mass (SRM) testing results are available.
- The dilation angle is determined through a ratio (dilation angle/friction angle) that is determined as a function of the GSI of the rock mass and multiplied by instantaneous friction angles in the model. The non-linear nature of the peak failure envelope and the associated instantaneous friction angles in this model result in higher dilation angles at lower confinement, and lower dilation angles at higher confinement. This behaviour is similar to that observed in laboratory rock testing results.
- The dilation angle is also mobilised and softens with increasing plastic shear strain in the model and drops to its residual value (30 per cent of the instantaneous friction angle) when the critical strain limit is exceeded. The basis for the dilation angle calculations is largely derived from Zhao and Cai (2010) and Lorig and Varona (2013).
- IUCM accounts for the confinement dependency of the rock damage process at various stages from peak to residual, as previously described. Confinement dependency is probably one of the most important factors controlling rock damage; however, it is largely ignored in most conventional constitutive models.
- Most conventional constitutive models assume a constant modulus of elasticity for the rock mass, irrespective of its damage state. In real-life situations, when rock

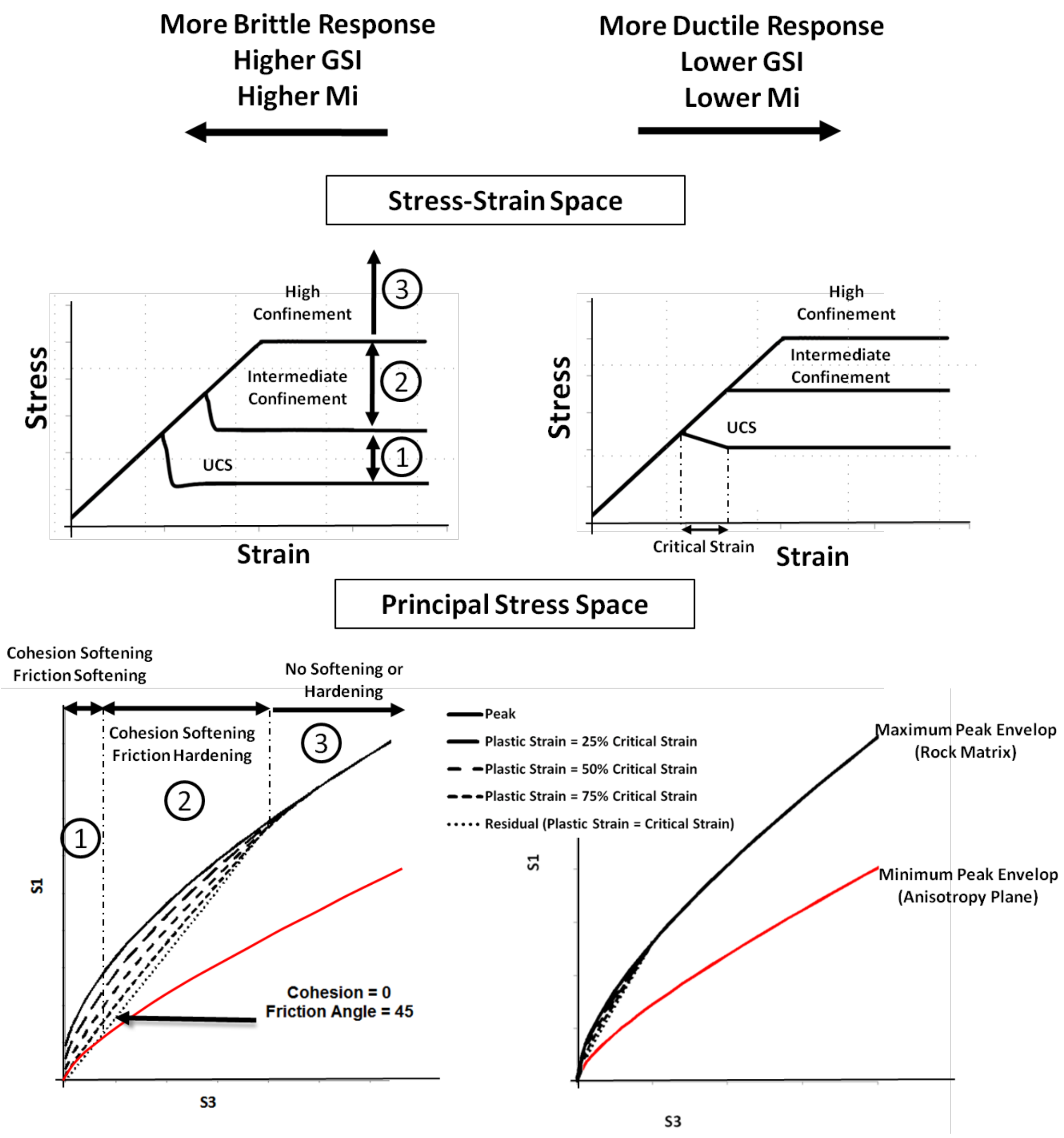

FIG 4 – A conceptual representation of different components of the improved unified constitutive model.

undergoes failure and continuous loading, more voids are generated within the rock mass. The rock mass porosity is subsequently increased. The greater the porosity in the rock mass, the smaller its elastic modulus. The drop in rock mass modulus can significantly affect the redistribution of stresses around a failed area and the subsequent phases of induced confinement. The impact of modulus softening can be more pronounced in situations where significant rock mass yield or deformation is expected, for example in high-stress conditions, caving, or deep open pit mining.

Reyes-Montes *et al* (2012) reviewed and collated the previous literature in this area and presented an empirical relationship between modulus drop and the level of porosity in a rock mass. IUCM uses this relationship to update the elastic modulus values according to new porosity levels. The porosity is calculated using the model volumetric strain outputs. The density in the model is also updated as a result of new porosity levels.

- The strength anisotropy in IUCM is explicitly included through a ubiquitous joint model, which accounts for both rock matrix strength and the lower strength associated with the existence of an anisotropy plane. For anisotropic rocks, the model uses two non-linear Hoek–Brown failure envelopes. One envelope defines the maximum strength and is related to the rock matrix strength. The other defines the minimum strength associated with the anisotropy plane.
- This constitutive model is implemented in the explicit finite difference code FLAC3D (Itasca Consulting Group, 2009) and therefore uses a time-stepping solution for calculations. As a result, progressive and time-dependent failure can be replicated in this model through updating the material properties as a function of new confinement and strain levels.

The key input parameters that are used in IUCM for anisotropic rock are:

- elastic modulus of intact rock
- UCS of intact rock ($\sigma_{c\ max}$)
- anisotropy factor ($\sigma_{c\ max}/\sigma_{c\ min}$)
- $m_{i\ min}$
- $m_{i\ max}$
- GSI.

All of these parameters can be determined from laboratory testing as described earlier in this paper and from core logging or structural mapping.

If, due to lack of sufficient and appropriate testing, the anisotropy factor and $m_{i\ min}$ cannot be determined, the information in Table 1 should be used to assign an indicative anisotropy factor. In addition, a $m_{i\ min}$ value equal to 50 per cent of $m_{i\ max}$ should be adopted.

The GSI value should ideally be estimated from field observations; however, if this is not possible, it should be estimated from joint condition and Rock Quality Designation values and according to relationships proposed by Hoek, Carter and Diederichs (2013). The formulae for conversion of Rock Mass Rating and Q′ values to GSI is not recommended.

APPLYING THE IMPROVED UNIFIED CONSTITUTIVE MODEL IN SENSITIVITY ANALYSES

Using the IUCM and the parameters of a calibrated base model, the authors conducted sensitivity analyses to better understand the relationship between the various properties of anisotropic rocks and the degree of resulting damage.

Closure strain, depth of cracking (dc), and volumetric strain were used to represent damage for each modelled case. As shown in Figure 5, volumetric strain can be used as an indicator of the degree of disintegration in continuum numerical models.

The results of these sensitivity analyses were used to derive the damage classification matrices shown in Figures 6 to 9 for vertical and horizontal development in anisotropic rock mass conditions. It should be noted that for the horizontal development scheme, medium anisotropy (anisotropy factor = 3) was assumed in all cases. Different responses are expected for conditions with a greater or lesser degree of anisotropy.

The following are the key conclusions of the sensitivity analysis:

- Both the degree of anisotropy and the orientation of the anisotropy plane with respect to stress and excavation walls have a significant impact on the stability of excavations and the damage propagation into the rock mass.

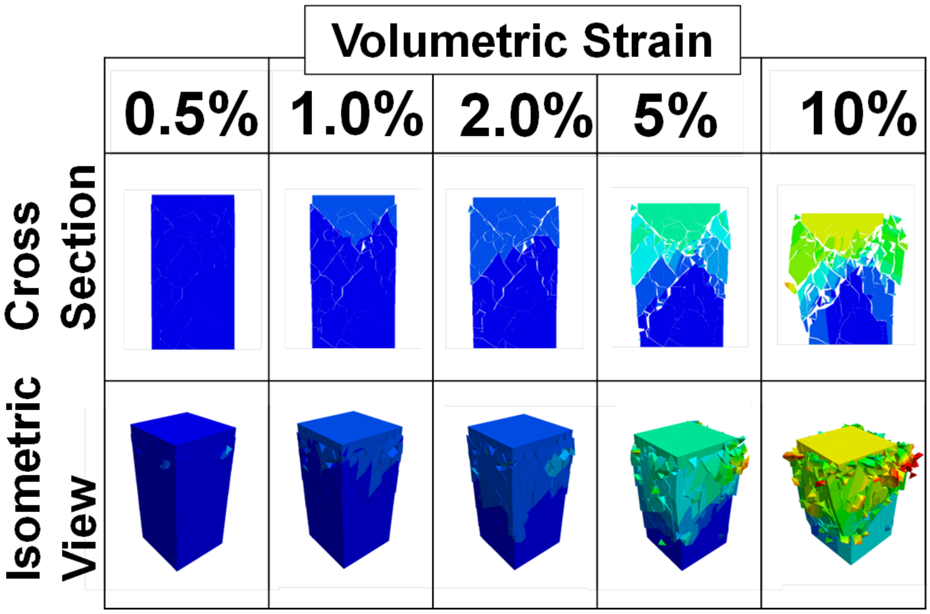

FIG 5 – Visual representation of degree of rock disintegration at various levels of volumetric strain.

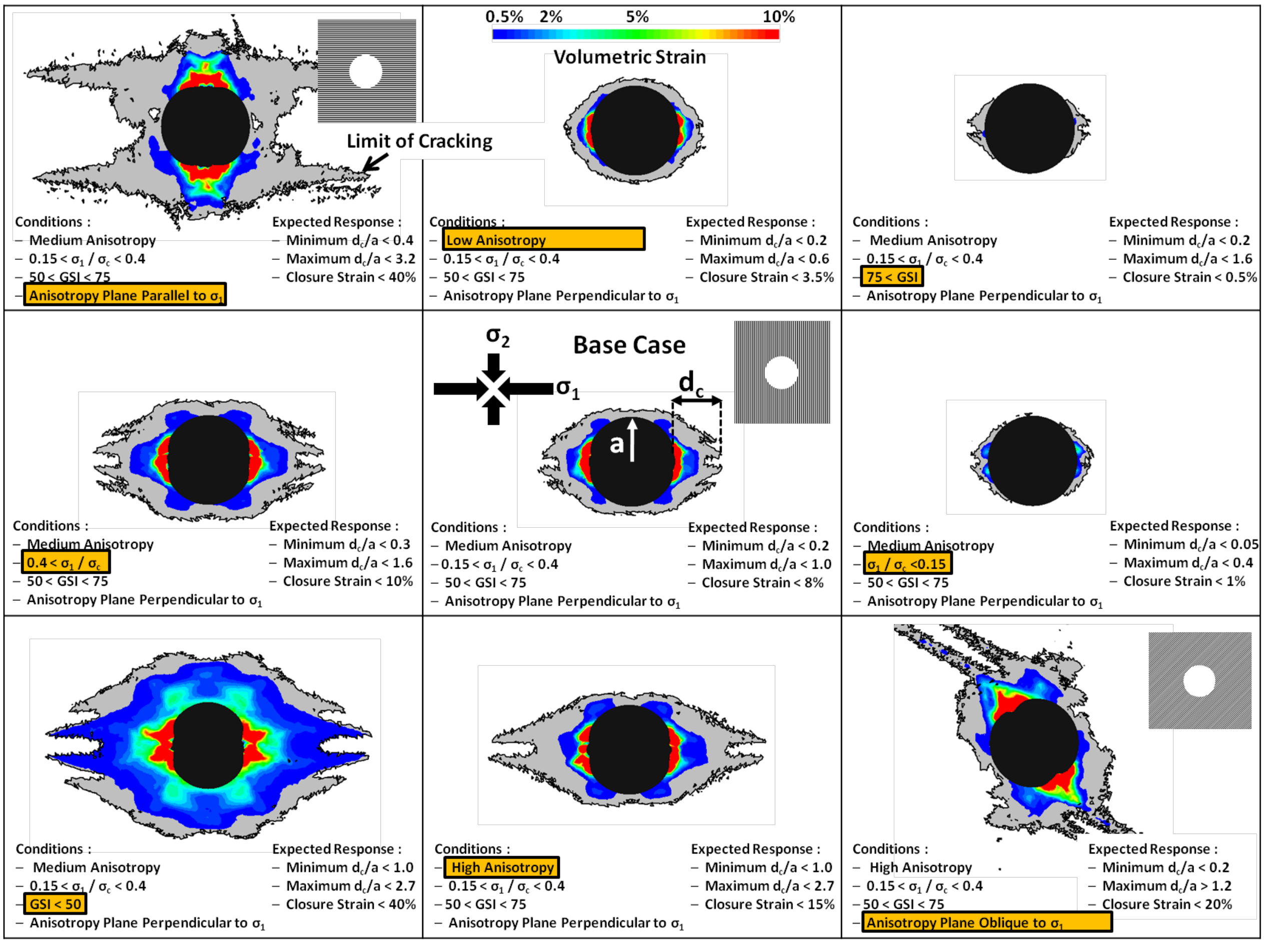

FIG 6 – Response of anisotropic rock mass to vertical underground development and sensitivities to main inputs.

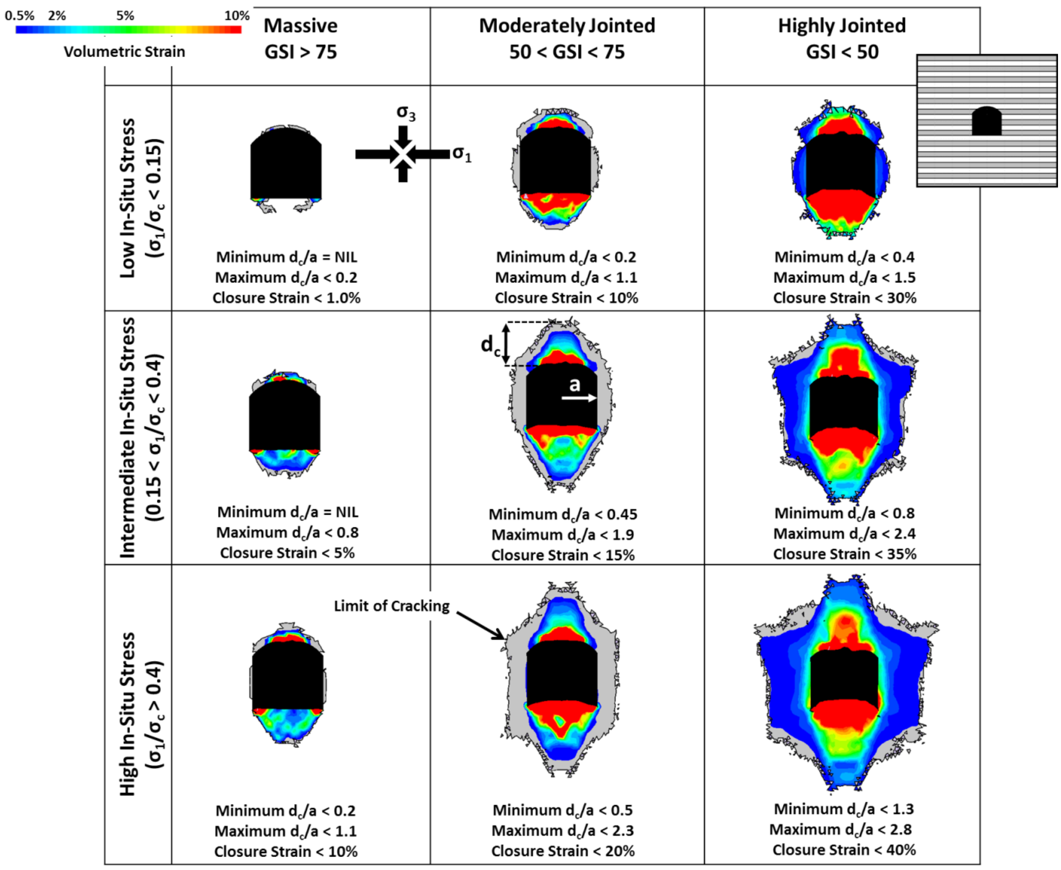

FIG 7 – Response of anisotropic rock mass to horizontal underground development (flat dipping anisotropy plane).

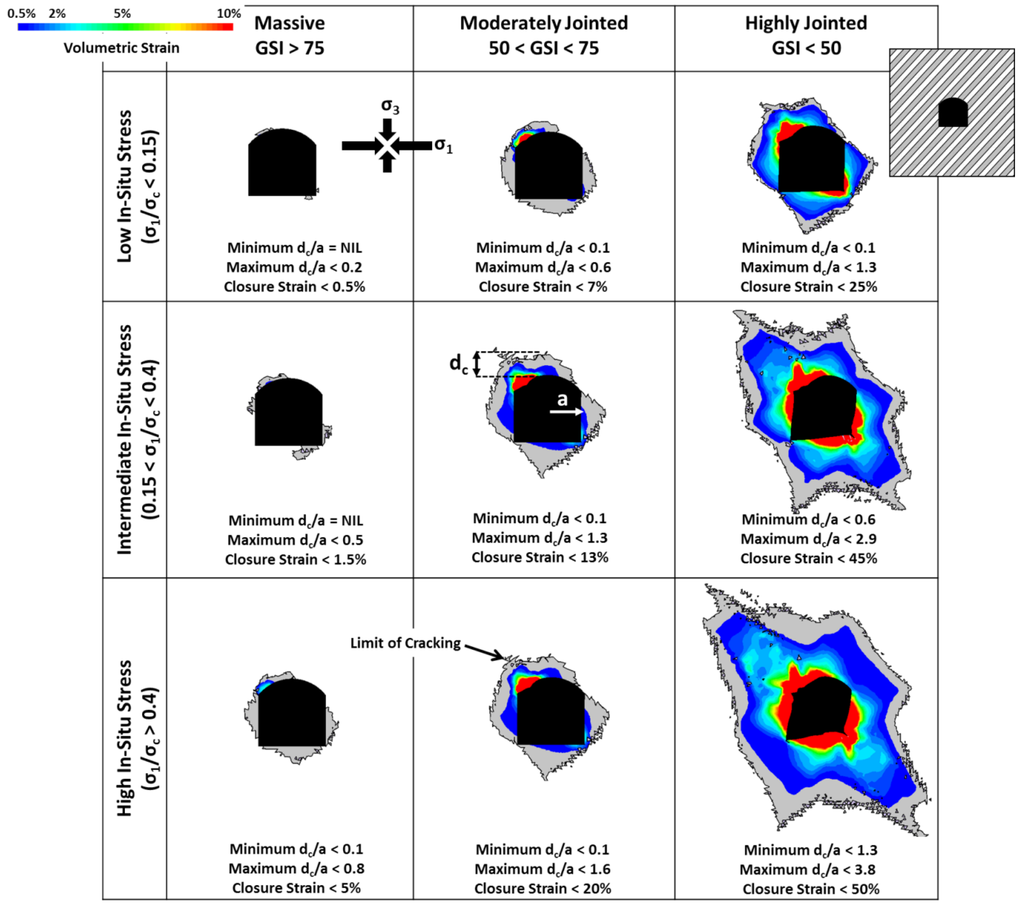

FIG 8 – Response of anisotropic rock mass to horizontal underground development (steeply dipping anisotropy plane).

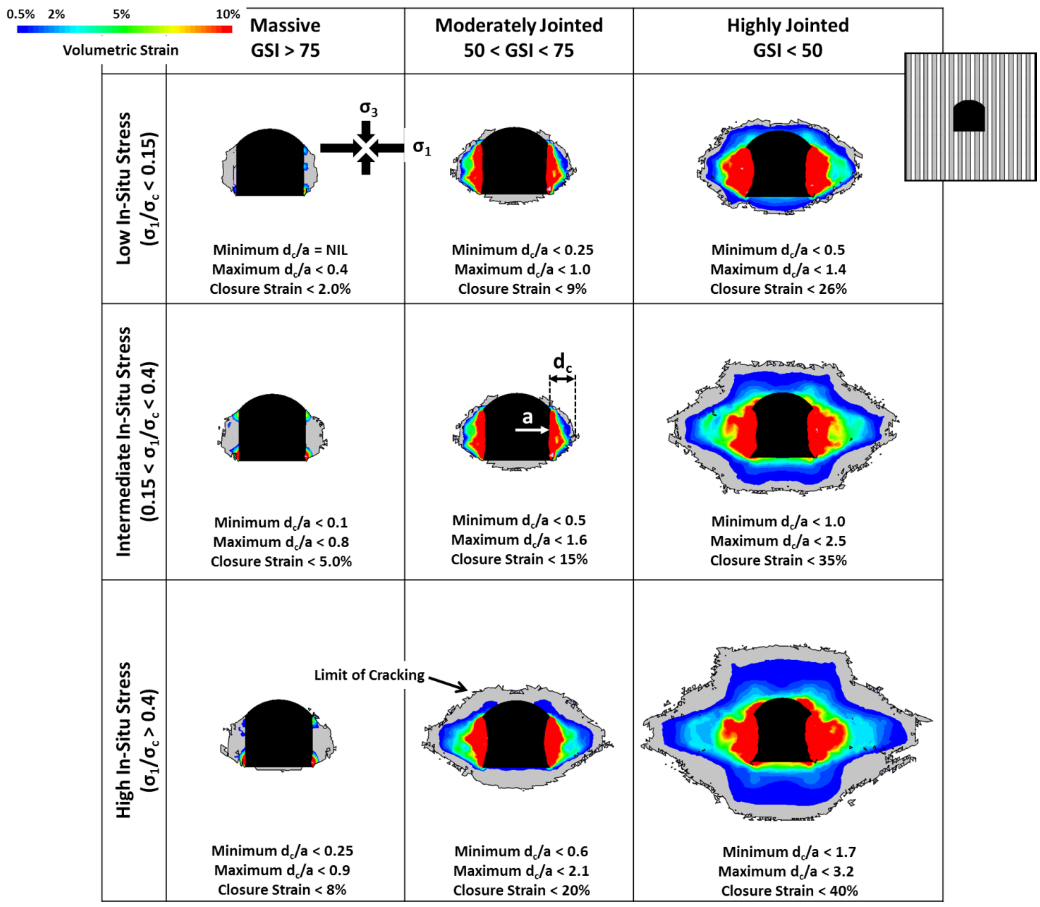

FIG 9 – Response of anisotropic rock mass to horizontal underground development (subvertical anisotropy plane).

- The depth of fracturing or depth of failure in anisotropic rocks can be significantly different to that in isotropic rocks. Therefore, conventional methods for depth of failure prediction (for example, Martin, Kaiser and McCreath, 1996) in isotropic rock conditions should not be applied to anisotropic rocks. Comparison between the results presented here and those presented by Vakili *et al* (2013) for isotropic rocks also confirms this large discrepancy.
- The same discrepancy exists between the closure strains presented here and those recommended by others including Hoek (2001), Lorig and Varona (2013) and Vakili *et al* (2013).

For all the modelled cases shown in Figures 6 to 9, it was assumed that the drive or shaft was excavated in a direction subparallel to the anisotropy plane. As was shown by Hadjigeorgiou *et al* (2013), development that is subparallel to the anisotropy plane will experience increased rock mass damage (or increased closure strain [ε]) compared to development that intersects perpendicular to the anisotropy plane. Figure 10 shows this concept and the results for the base case numerical model. It should be noted that the closure strain and its orientation dependency can be significantly different to the examples shown in Figure 10 when different stress conditions or degrees of anisotropy exist.

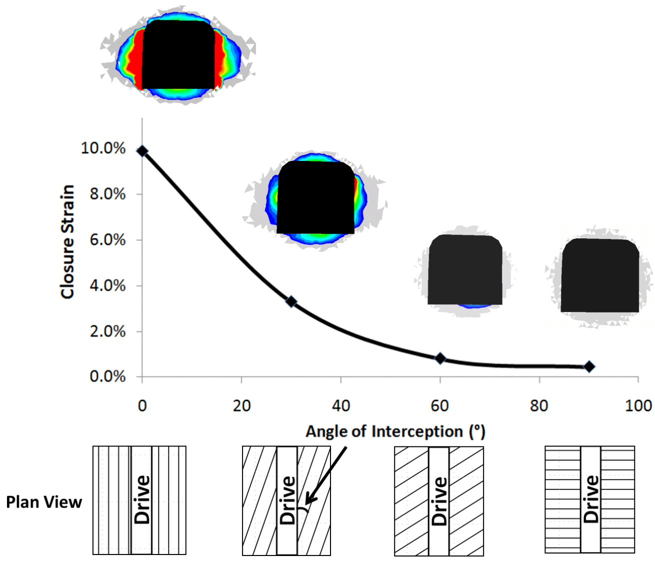

FIG 10 – Influence of angle of interception on closure strain (ε).

CASE STUDIES

To demonstrate the suitability and validity of the guidelines presented in this paper and the IUCM, two case studies are presented here.

In both cases, the improved guideline and IUCM are compared against the conventional analysis technique, which uses an isotropic linear Mohr–Coulomb failure criterion. In this conventional technique, the average uniaxial compressive strength of intact rock ($[\sigma_{c\,max} + \sigma_{c\,min}]/2$) or minimum strength ($\sigma_{c\,min}$) are used to derive the rock mass properties. For the case studies presented here, it was assumed that $\sigma_{c\,min}$ was applied.

Case 1 – vertical and horizontal development in 'very high anisotropy' conditions

For this case, rock mass response and the observed damage associated with two excavations in highly foliated rock mass conditions were used to evaluate the suitability of the proposed techniques and compare them with the conventional methods.

The first excavation was a vertical shaft excavated using a raise-boring technique. As shown in Figure 11, shortly after excavation, significant buckling was observed in the east and

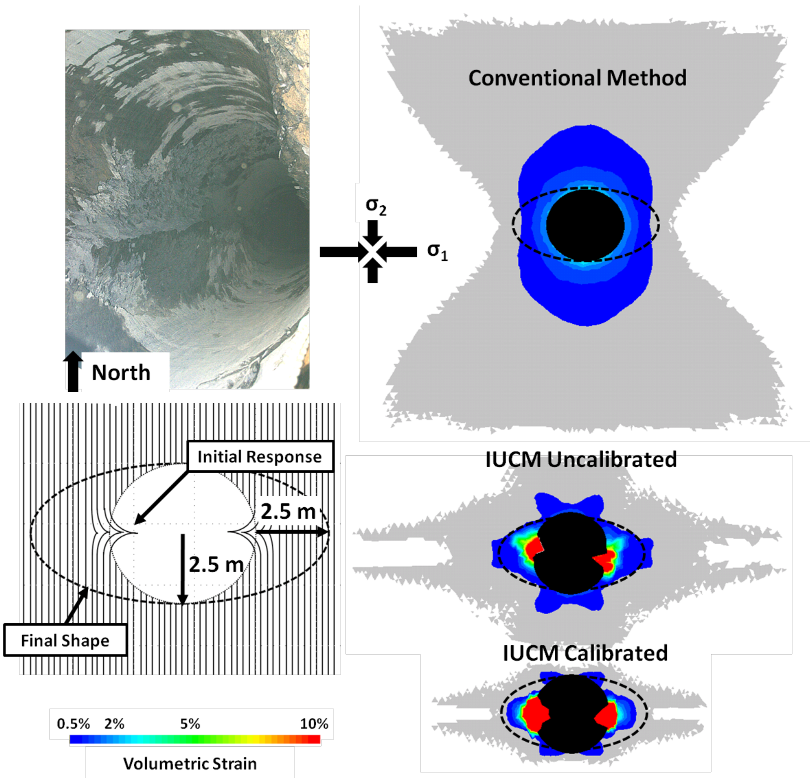

FIG 11 – Comparisons between predictions of the improved unified constitutive model and conventional method and the actual response of the rock mass for the vertical development in case 1.

west walls of the shaft. Ultimately, the east and west walls experienced up to 2.5 m of overbreak.

The second excavation was a horizontal drive developed in similar ground conditions to the shaft and with its orientation (drive axis) subparallel to the foliation planes. This excavation was supported using surface support and reinforcement elements. Monitoring instruments were installed in the back and sidewall of this drive and displacement was carefully measured during the cyclic excavation of the drive. This was done to capture the increased deformation levels as a result of subsequent face advances (see Figure 12). The exact firing (face advance) dates could not be sourced to match with the reported deformations; however, it was known that 4 m face cuts were implemented by the mine site.

Tables 3 and 4 show the input parameters that were adopted for the numerical analysis. The initial (uncalibrated) inputs were derived from geotechnical logging data, structural mapping data and laboratory testing according to procedures explained earlier in this paper. Stress measurement data was used to derive the stress state in this case. No triaxial testing was conducted in this case. The $m_{i\ max}$ value was estimated from the published information and was halved to obtain the $m_{i\ min}$ value. The modified (calibrated) parameters were derived as a result of an iterative back-analysis study, which was conducted to better match the model outputs and the field observations.

TABLE 3
Input parameters used in case 1 for the improved unified constitutive model and comparison between calibrated and initial (uncalibrated) inputs.

Input parameter	Initial (uncalibrated)	Modified (calibrated)
$\sigma_{c\ max}$ (MPa)	185	185
Geological strength index	58	55
$M_{i\ max}$	7 (estimated)	30
Ei (Gpa)	50	50
Anisotropy factor	6	10
Foliation dip (°)	85	85
Foliation dip direction (°)	270	270

TABLE 4
Stress state used in case 1 and comparison between calibrated and initial (uncalibrated) stress.

	Initial (uncalibrated)		
Stress components	Magnitude (MPa)	Dip (°)	Bearing (°)
σ_1	60	15	278
σ_2	40.5	64	41
σ_3	33.5	20	182

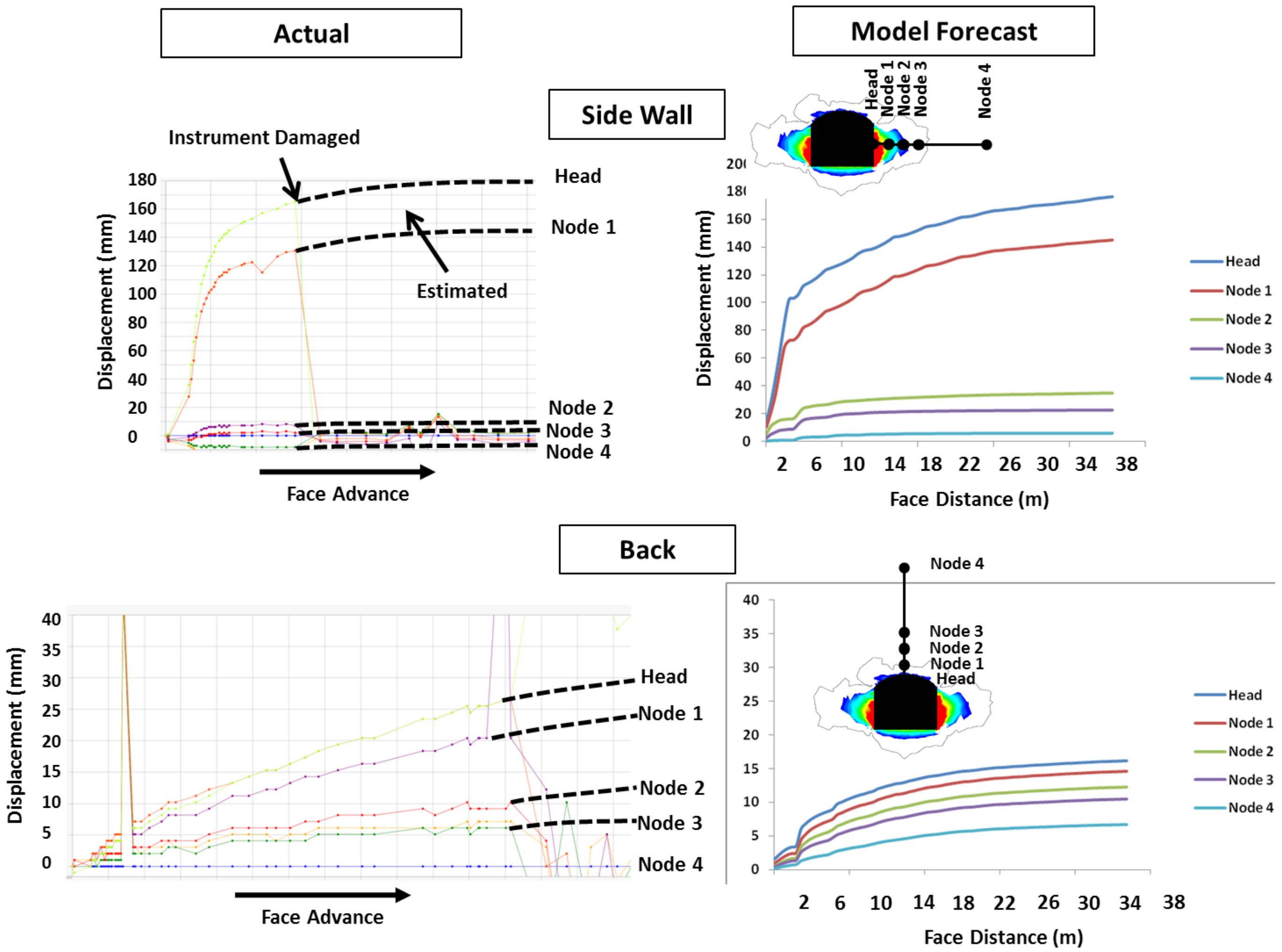

FIG 12 – Comparisons between displacement forecasts by calibrated improved unified constitutive model and actual displacement monitoring data for the horizontal development in case 1.

Figure 11 shows the comparison between the observed response and the model predictions for the vertical shaft. The position, extent and severity of damage forecasted by the conventional method is significantly different to actual observations; however, the model using IUCM forecasted the damage with reasonable accuracy, even prior to any calibration.

The calibration effort in the model that used IUCM was focused on matching the overbreak and extent of buckling in the vertical shaft as well as the measured displacement in a horizontal drive. Figure 12 shows the comparison between actual displacement measurements (extensometer data) and the calibrated model outputs for the subject development drive. It should be noted that the ground support systems were explicitly included in the calibrated model.

This calibrated model was later utilised to assist with optimisation of ground support systems in the mine site under study.

Case 2 – open stoping in 'medium to high anisotropy' conditions

For this case, information regarding observed overbreak in an open stope in a moderately foliated rock mass was used for validation purposes. The stope under study was located in a relatively isolated location with some minor nearby excavations.

Tables 5 and 6 show the initial uncalibrated input parameters and the subsequent modified calibrated parameters. Similar to case 1, the initial inputs were derived from field and laboratory test data.

After excavation of the stope, significant hanging wall overbreak (to a depth of up to 17 m) was recorded. The final stope geometry obtained from cavity monitoring system (CMS) surveys is shown in Figure 13.

The model that used the IUCM was able to forecast the extent, mechanism and severity of damage with a reasonable level of accuracy, even prior to any calibration. The subsequent model calibration was conducted to better match the model forecasts with the observed overbreak geometry. A volumetric strain limit of three per cent in the calibrated model resulted in a close match with the actual overbreak geometry.

The conventional method in this case underestimated the extent of damage considerably and was not able to replicate the observed overbreak geometry.

The model provided a means to more accurately assess the expected extent of hanging wall overbreak. This led to the application of cable bolts to manage the near field fissility of the stope boundaries, which significantly reduced overbreak and subsequent stope dilution.

CONCLUSIONS

Despite the concept of anisotropy being well-understood, many geotechnical practitioners still use methods of analysis that were designed for isotropic rock mass conditions only; however, to enable sensible prediction of failure mechanisms and the location, severity and extent of damage, analysis techniques should account explicitly for both rock matrix strength and strength along the anisotropy planes.

When dealing with anisotropic rocks, specific procedures and guidelines should be followed for data collection and selection of input parameters. The procedures and guidelines presented in this paper should enable a better understanding of strength anisotropy and assist with selecting input parameters for subsequent analysis.

TABLE 5

Input parameters used in case 2 for the improved unified constitutive model and comparison between calibrated and initial (uncalibrated) inputs.

Input parameter	Initial (uncalibrated)	Modified (calibrated)
$\sigma_{c\,max}$ (MPa)	130	100
Geological strength index	65	60
$M_{i\,max}$	22	20
Ei (Gpa)	50	50
Anisotropy factor	3	5
Foliation dip (°)	40	35
Foliation dip direction (°)	90	70

TABLE 6

Stress state used in case 1 and comparison between calibrated and initial (uncalibrated) stress.

Initial (uncalibrated)			
Stress components	Magnitude (MPa)	Dip (°)	Bearing (°)
σ_1	60.5	02	348
σ_2	36.6	56	255
σ_3	22.2	33.6	079
Modified (calibrated)			
σ_1	60.5	02	348
σ_2	36.6	33.6	079
σ_3	22.2	56	255

The IUCM briefly described in this paper can account for some fundamental failure processes such as strain-softening, confinement-dependency, dilatational response, stiffness softening mechanisms, anisotropic behaviour, progressive damage and time-dependency. These factors, which are often ignored in most conventional analysis techniques, play an important role in more complex ground conditions.

The sensitivity analysis conducted for this paper showed that the degree of anisotropy and orientation of the anisotropy plane with respect to stress and excavation walls can have a significant impact on the stability of excavations and the damage propagation into the rock mass. This can even override other important geotechnical factors, such as stress, in controlling the resulting failure mechanisms.

Closure strain values and the depth of failure in anisotropic rocks can be significantly different to those in isotropic rocks. Conventional methods for prediction of depth of failure and closure strain should not be applied for anisotropic rocks.

Two case studies were presented in this paper to validate the suitability of the IUCM and the proposed procedures and guidelines. The results show that the proposed methods are able to provide reasonable estimates of the likely failure mechanisms and their severity even without any calibration efforts. This is of course subject to the soundness of the initial input parameters obtained from laboratory testing and field investigations. When calibration information exists, minor adjustment to inputs would typically be required to better match the model outputs with the monitored responses.

It was also shown that application of the linear isotropic Mohr–Coulomb failure criterion and other conventional design methods can lead to significant errors and should be avoided in more complex anisotropic conditions.

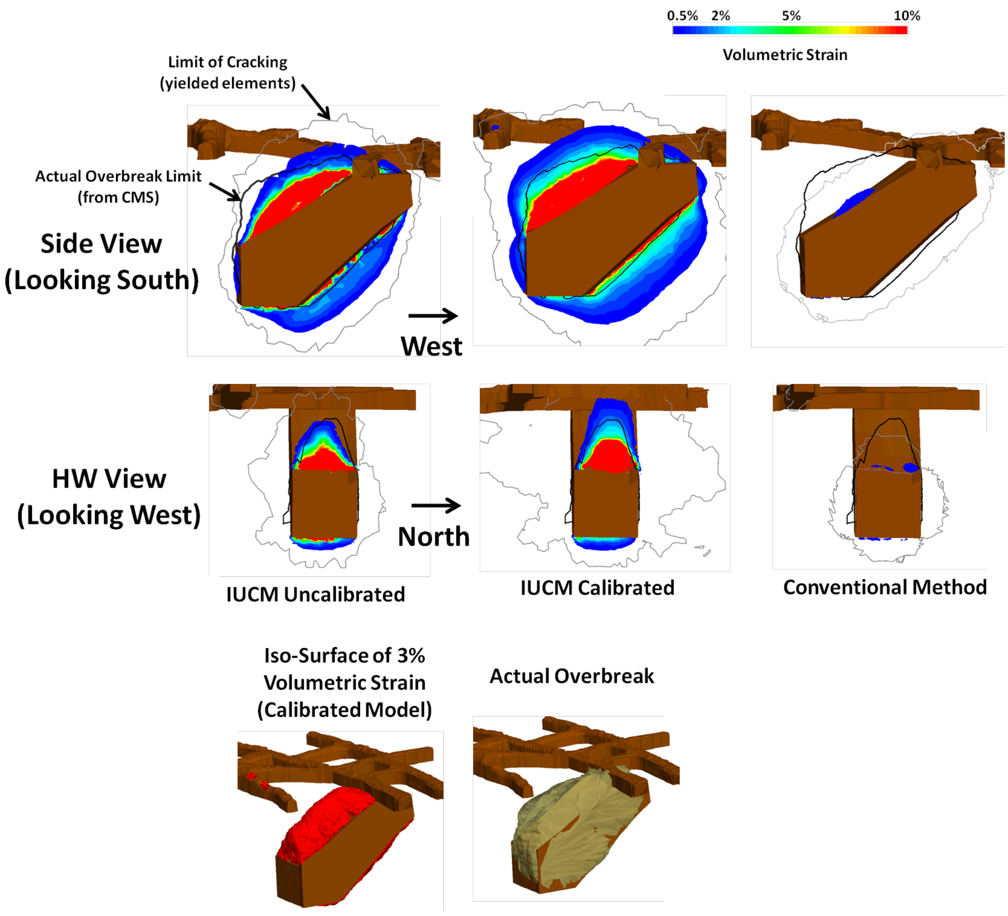

FIG 13 – Comparisons between predictions of the improved unified constitutive model and conventional method and the actual response of the rock mass for the open stope in case 2.

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