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p. 483-489

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ABSTRACT: A procedure is presented for the design of reinforcement for highly stressed rock based on 3D numerical stress analysis using the MAP3D code. Modelling requires extensive characterisation of the rock mass strength and deformability and appropriate characterisation of the stress field. The numerical model is calibrated using a Rock Mass Damage Criterion and a Rock Mass Failure Criterion that are calibrated to observations of in situ cracking. These criteria define an outer damaged or cracked zone and an inner, failed or broken zone. Examples are used to show how the extent and dimensions of these zones can be determined by post-processing the modelling results and how the boundary of these zones can be used to dimension the primary reinforcement scheme.

1 INTRODUCTION

The effectiveness of a particular reinforcement scheme in terms of its density and length can be assessed using empirical strategies, theoretical methods and geotechnical instrumentation (Windsor and Thompson, 1993). In addition, detailed stability analysis may now also include the calibration of a numerical model program to simulate the excavation sequences in order to assess the role of stress change on the rock mass environment. Three dimensional numerical modelling results can then be used to determine the overall stability around underground excavations, where zones of damage, or failure can be estimated and then used to determine the required length and capacity of a reinforcement scheme. The required input parameters consist of the in-situ stress profile with depth, the strength and deformational properties of the rock mass as well as the excavation steps and their sequence.

2 ROCK MASS STRENGTH & DEFORMABILITY

The rock mass compressive strength is a measure of the peak load carrying capacity of a rock mass. It is defined as a proportion of the intact rock strength due to the presence of geological discontinuities (Hoek and Brown, 1980). The rock mass strength is defined here as the limiting load required for stress driven failures to initiate and propagate around underground excavations. In modern engineering mine design, the rock mass strength is usually estimated prior to excavation using borehole data (i.e. as part of the orebody delineation process, when full core is available) to determine the variability of the intact rock properties and the rock mass classification parameters throughout the orebody. The intact rock parameters (uniaxial compressive strength and the elastic constants E and ν) can be determined from a number of representative holes taken from the full set of exploration holes (Figure 1) thus allowing the characterization of the entire orebody.

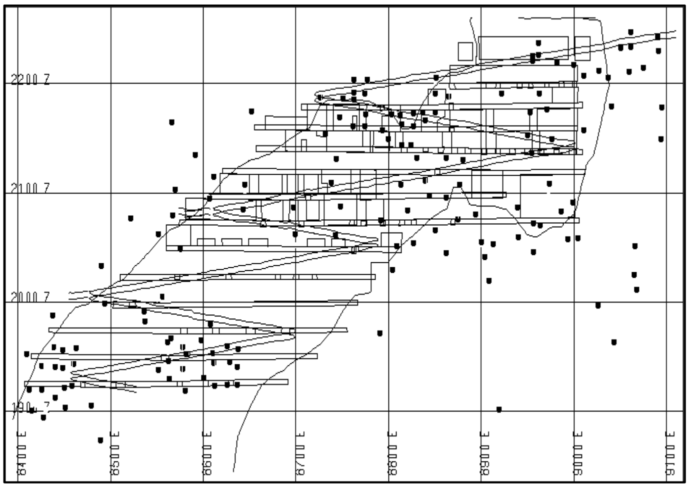

Figure 1. Typical distribution of exploration holes in an orebody.

Usually, the uniaxial compressive strength and its variation for each rock type present can be defined such as in Table 1. Alternatively it may be presented using modelled contours across a particular unit such as the strength distribution in the hangingwall boundary as shown in Figure 2.

Table 1. Average Uniaxial Compressive Strength per rock type.

Rock Type	Sample Number	UCS	STDev
		(MPa)	(MPa)
Hangingwall rock	9	114	25.1
Orebody	11	107	42.2
Footwall rock A	7	139	40.1
Footwall Rock B	15	107	23.0

Figure 2. Modelled UCS variability across an orebody hangingwall boundary.

The actual value of rock mass strength and deformability depends upon the geometrical nature and strength of the geological discontinuities, which can be estimated by using empirical methods that rely on rock mass classifications. Table 2 presents some typical average results for the same rock units described before in Table1.

Table 2. Average rock mass properties per rock type.

Rock mass Type	E_m	σ_{cm}	ϕ_m
	(GPa)	(MPa)	(°)
Hangigwall Rock	32	46	40
Orebody	31	44	44
Footwall Rock A	32	57	40
Footwall Rock B	31	44	44

Where: E_m is the deformation modulus, σ_{cm} is the uniaxial compressive strengths, and ϕ_m is the friction angle of the rock mass.

3 IN SITU STRESS

Reliable evaluation of *in situ* stress is an important phase in the analysis and design of underground excavations, particularly when evaluating excavation stability with the aim of preventing stope/pillar wall failures. Consequently, over the last seventy years considerable effort has been invested by numerous research organizations in finding suitable methods to quantify Earth's crustal stresses. In cases where excavation access is available, the overcoring method using the CSIRO HI cell has proven to be an accurate and reliable method of measuring the complete 3D stress tensor. In addition, in the last few years the Western Australian School of Mines (WASM) has developed a technique to determine the complete 3D stress tensor using the Acoustic Emission method (Villaescusa et al, 2003). The practical advantages of the WASM AE technique revolve around the fact that the state of stress may be quantified for any point where oriented exploration core can be obtained. This negates the previous restriction for the existence of an excavation from which to conduct the measurements.

The results shown in Figure 3 compare the stress tensors obtained by the CSIRO HI cell and the WASM AE method for the orebody shown in Figures 1 and 2. The comparison involves a two point WASM AE stress profile defined over a 100m interval and a single CSIRO HI overcoring result at a shallower depth separated by 150m. The principal stress magnitude-depth relationships and the principal stress orientations for the two point WASM AE profile and the single CSIRO HI result are given together in Figure 3.

The data shows that there is excellent agreement between the extrapolation of WASM AE magnitudes of the principal stresses compared with those obtained by the CSIRO HI cell overcoring and between the principal stress orientations indicated by overcoring at 363m depth compared to that obtained by WASM AE at 493m depth. Comparison of the two WASM AE measurements at 493m and 595m indicate a small rotation that effectively 'flips' the major and intermediate principal directions to diametrically opposite positions on the projection. The relative variation for each principal stress magnitude is shown in parenthesis under each projection of WASM AE principal orientations.

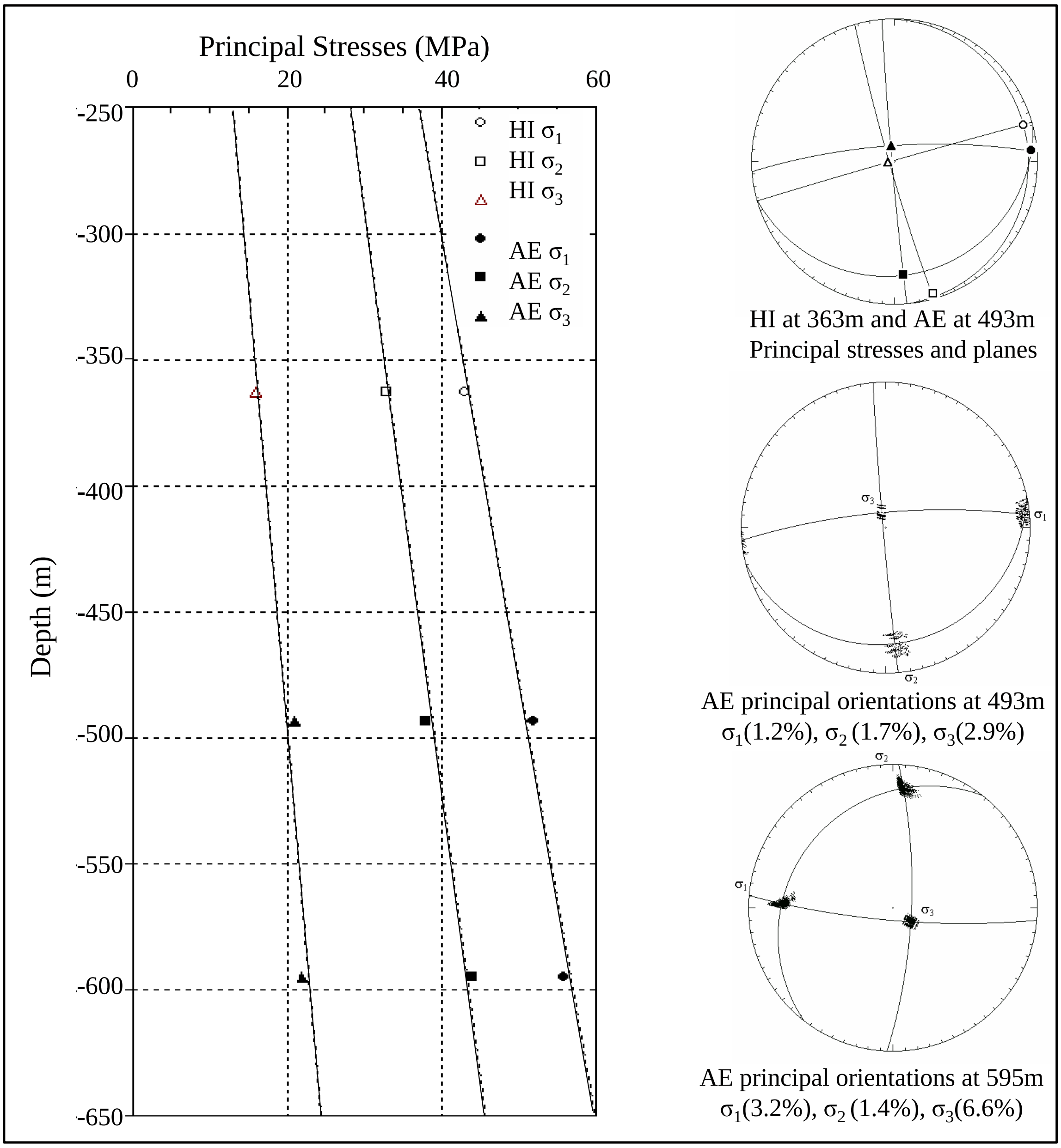

Figure 3. Comparison of a three point AE stress measurement profile with a single point CSIRO HI Cell stress measurement.

4 NUMERICAL MODELLING

In most cases of underground mining, the induced stresses may be determined using linear elastic numerical modelling. The required inputs are the in-situ stress field with depth, the estimated deformational properties of the rock mass and the chosen extraction sequence. In this study the elastic version of the computer program Map3D was used to determine the stress distribution around the underground excavations. It must be understood that the results are used in conjunction with structural information (for example large fault behaviour) in order to interpret any excavation option analyzed as well as their respective reinforcement strategies. Typical output from numerical modelling includes stresses and displacements. These can then be compared with empirical failure criterion established for the different domains around the underground excavations within an orebody. It must be emphasized that any predictive models must be calibrated (validated) against field data and observations using either visual methods and/or geotechnical instruments.

Although linear elastic modelling can be used to estimate the level of damage and the extent of the failure zones, non-linear models are required to simulate any resultant stress re-distribution from such failures. Progressive orebody extraction may induce several phases of post-peak behaviour in a rock mass and similarly, small changes to the stress field induced by distant extractions may induce significant rock mass damage around a particular excavation wall.

5 FAILURE CRITERION

Experience through correlation of underground observations and geotechnical instrumentation with numerical modelling results suggest that a rock mass is damaged when a range of induced stress levels exceeds a certain site dependent threshold as shown in Figure 4. Below the damage threshold the response is elastic and usually very little damage can be observed. However, with increased overstressing increased damage is experienced. The actual damage level reached depends upon the amount of overstressing and beyond the initial damage threshold a zone of potential overbreak (POB) is reached. Increased stress beyond this level may cause stress driven failures and eventually the rock mass may become unsupportable.

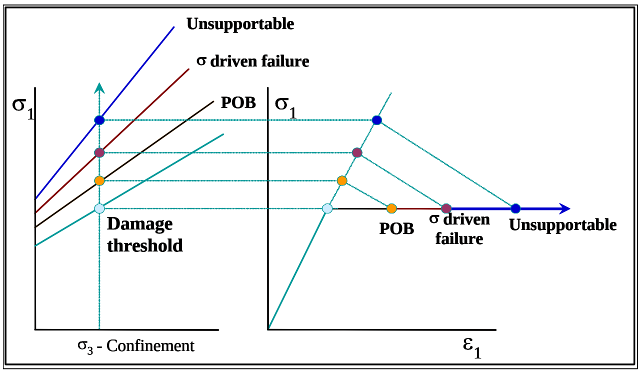

Figure 4. Different levels of stress driven damage and failure.

Consequently, a rock mass is neither strictly failed nor unfailed, but rather for similar confinements, there is a range of stress levels where increasing excavation damage is experienced. A Rock Mass Damage Criterion can be defined as follows:

$$\sigma_1 = A + p\,\sigma_3 \qquad (1)$$

where A and p are site dependent constants. Back analysis of numerical modelling over a number of years (Wiles 1998, 2004) suggests that p normally takes on a value near unity.

Figure 5 shows an empirical damage criterion established from back analysis of cavity monitoring system (CMS) surveys for a primary stope located at a deep underground operation in Western Australia. The criterion expressed by Equation 1 is also conceptually represented by the lower line in Figure 6 and can be interpreted to represent the stress level where seismicity is observed to occur. In addition, borehole cameras can be used to directly observe the amount of damage in the form of increased fracture frequency. Geotechnical instrumentation shows that when this stress level is exceeded a loss of rock mass cohesion is experienced. However, a considerable degree of residual frictional strength (i.e. interlocking) is still available. Nevertheless, the rock mass is visibly cracked and may unravel and disintegrate if it is not held together by a ground support scheme.

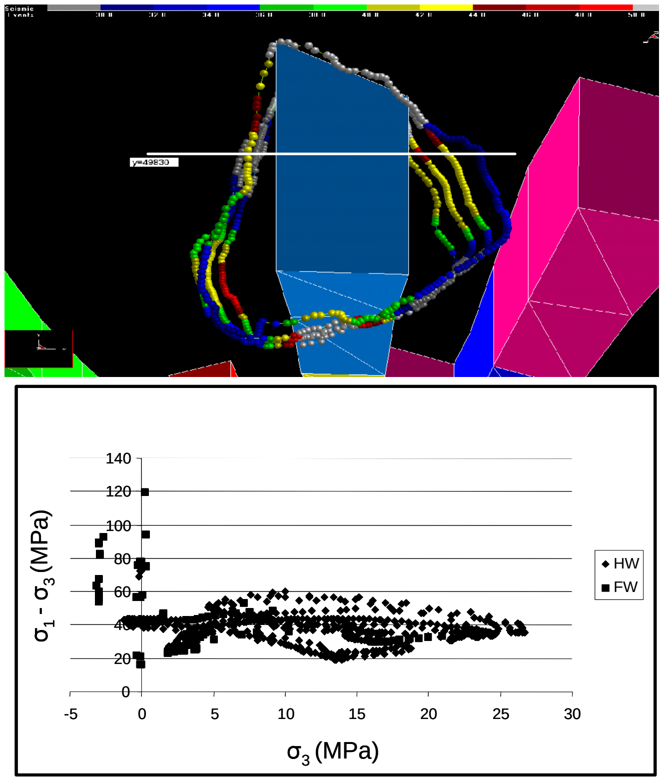

Figure 5. CMS profile of failure and damage criterion from back analysis using Map3D.

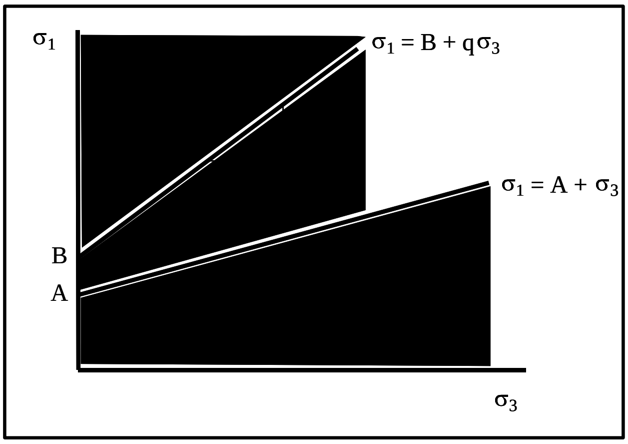

Figure 6. Rock mass damage zones

Another criterion that can be readily identified (upper line in Figure 6) is commonly called the Rock Mass Strength Criterion and is defined as follows:

$$\sigma_1 = B + q\,\sigma_3 \qquad (2)$$

where B is the Uniaxial Compressive Strength (σ_{cm}) of the rock mass and q is related to the rock mass friction angle (Φ_m) by $\tan^2(45 + \Phi_m/2)$.

$$\text{Strength Factor A} = \frac{\sigma_{cm} + \sigma_3 \tan^2(45 + \phi_m/2)}{\sigma_1}$$

This criterion represents a stress level at which failures can be considered to be stress driven. When the stresses reach this level the interlocking is overcome and the rock mass undergoes considerable non-linear deformation. This deformation is driven by large forces that may not be held back by ground support schemes. In fact, ground support must be able to move with this deformation if the failed material is to be contained.

Data from a number of mines exhibiting brittle rock response suggests that A and B have similar magnitudes, and the two criterion may meet at σ_3=0. It is anticipated that this may not be true for more compliant rock types. In addition, the rock located within the zone defined between the two criteria can be considered to be damaged. As overstressing increases from the lower criterion to the upper one, the rock mass becomes progressively more sensitive, in that it is easier to trigger an unravelling failure, for example by blasting nearby.

While the rock mass strength failure criterion discussed above can be estimated by using empirical methods that rely on rock mass classification, correlation of underground observations and geotechnical instrumentation with back-analyses is used to verify whether the estimate is correct and refine the actual values.

For example in Figure 7, the two pillars towards the back were observed to fail right through to the core, while the pillar in the foreground experienced side wall spalling only. Elastic modelling can be used to determine the stress levels respectively in the core and side walls. These stresses can then used to verify the failure criterion. By repeating this type of back-analysis for many observations in situ, the site specific rock mass compressive strength representing stress driven failure can be determined as shown in Figure 8.

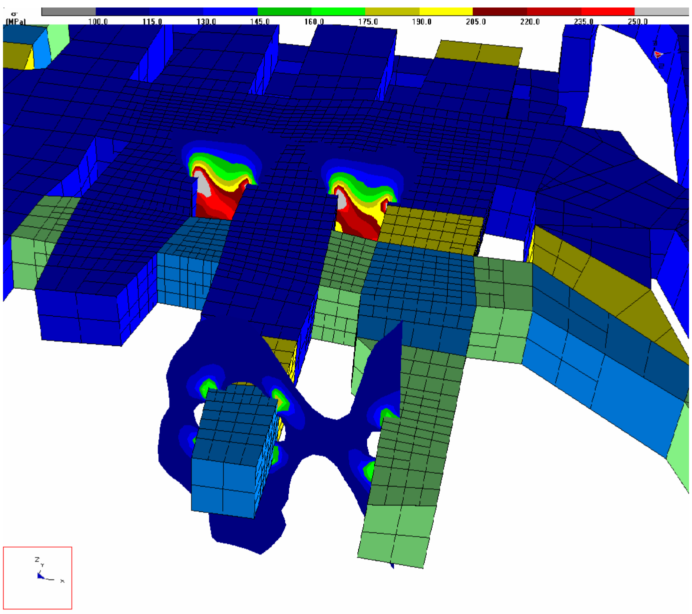

Figure 7. Modelled induced stresses in pillars using Map3D.

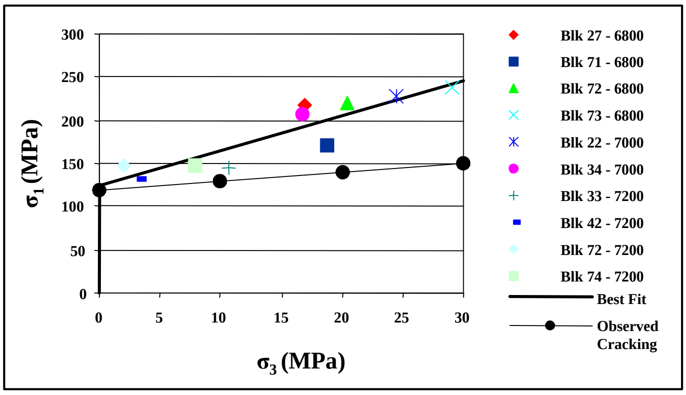

Figure 8. Back analysis of pillar failures.

Also shown in Figure 8 are results from a back analysis of locations where cracking was observed in boreholes. This provides values to be used in the failure criterion described above (Equations 1 and 2):

$\sigma_1 - \sigma_3 = 120$ Rock Mass Damage Criterion

$\sigma_1 = 124 + 4.1\,\sigma_3$ Rock Mass Failure Criterion

6 ROCK REINFORCEMENT DESIGN

Whether the strength parameters defined in Equations 1 and 2 are determined from back analysis, or estimated using empirical methods that rely on rock mass classifications, the design of the rock reinforcement in overstressed rock can be achieved using three dimensional numerical modeling . This can be achieved by assuming that the ground response can be described by two categories: broken ground and cracked ground as shown in Figure 9.

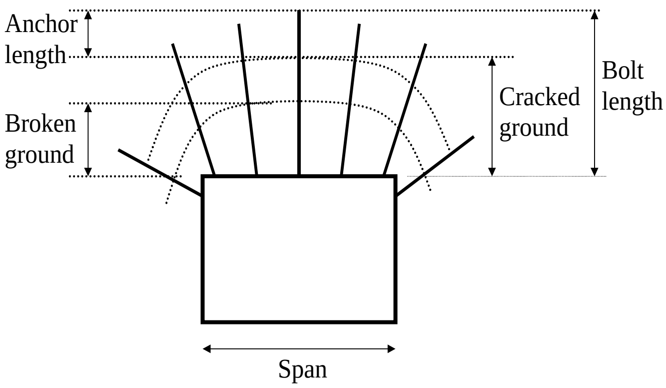

Figure 9. Zones used for rock reinforcement design.

The broken ground is ground that has undergone stress driven failure and represents the dead weight that our support needs to suspend. This will be determined using the rock mass stress failure criterion. Consider a highly stressed location shown in Figure 10. To determine the depth of broken ground, the values of strength divided by stress have been contoured, or $(124 + 4.1\ \sigma_3)/\sigma_1$. The results show that the broken ground depth extends 2 metres into the back.

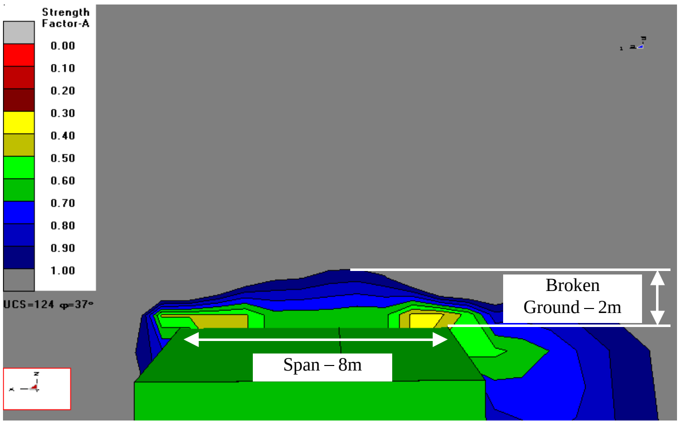

Figure 10. Contours of $(124 + 4.1\ \sigma_3)/\sigma_1$

The cracked ground defines where the reinforcement anchoring begins. This will be determined using the rock mass damage threshold criterion defined earlier. Consequently, the depth to the damage threshold is determined using the contoured values of $(\sigma_1 - \sigma_3)$ as shown in Figure 11.

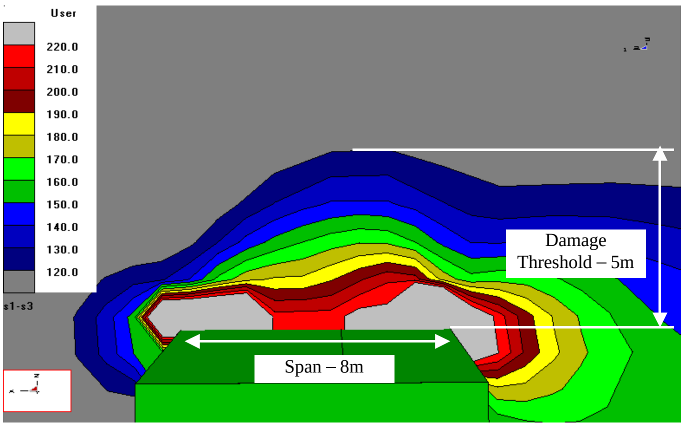

Figure 11. Contours of $(\sigma_1 - \sigma_3)$

In this location the dimensions and extent of the cracked or damaged zone (and within this zone the dimensions and extent of the broken or fractured zone) have been determined. The results suggest that cable bolts are required across the 8m span and will need to extend past the 5m deep damaged zone to be anchored within intact rock. The density of cable bolts may be determined by considering the mass of the damaged zone across the span of the opening. The type, stiffness and installation timing of the cable bolts chosen will depend on the expected velocity of loading, both for the current circumstance and for future mining induced stress changes. Investigation of the 2m deep broken or failed ground during the back analysis stage will indicate if and what type of rock bolts and mesh are required to retain the broken ground between the cable bolt array spans.

At other locations stress levels may be insufficient to induce stress driven failures.Therefore, at such locations this method would predict zero depth of broken ground and alternative failure mechanisms and alternative analysis methods depending on the structural geology and geometry of the openings must be considered. In medium to low stress conditions there are basically three cases of rock mass to consider: massive, stratified, and jointed rock. In massive rock a simple two dimensional, elastic analysis of the opening and the stress field may predict mild spalling as opposed to deep fracturing. In stratified rock, beam or plate theory may predict shearing and dilation to occur depending on the orientation of the stratigraphy and the orientation and shape of the opening. This may lead to bending and buckling with step path failures through the layers or cantilever action and guttering of the layers, which is common in coal mining collapse mechanisms. In jointed rock, block theory may be used to the stability of blocks of rock that may translate or rotate towards the opening. This mechanism may initiate with the loss of individual blocks but may propagate to a progressive collapse of the block assembly around the opening. In each case it will be necessary to predict the dimensions and extent of the failure zone and provide a reinforcement and or support scheme suitable for both global and local stability. The analysis methods and procedures for reinforcement design in these circumstances have been given by Hoek and Brown (1981) and Brady and Brown (1985).

Once an initial design has been formulated this modeling method may also be used to evaluate how other excavation stabilization techniques affect the rock reinforcement requirements. This would include modelling of alternative sequences, reduced spans and using backfill.

7 CONCLUSIONS

A procedure has been given for the design of reinforcement for highly stressed rock based on 3D numerical modelling using the Map3D code. The procedure involves a sequence:

1. Characterisation of the rock mass strength and deformability.
2. Characterisation of the stress field.
3. Definition of the mine geometry and excavation sequence.
4. Modelling of the stress redistribution due to excavation.
5. Back analysis to determine:
    a. A Rock Mass Damage Criterion
    b. A Rock Mass Failure Criterion
6. Post-processing of stored analysis results to define the outer, damaged or cracked zone and the inner, failed or broken zone.

7. Primary reinforcement is dimensioned on the geometry and mass of the damaged and failed zones.
8. Secondary reinforcement and or support is dimensioned on the geometry and likely behaviour of the failed zone local to the excavation surface.

This procedure is suitable for hard rock mines which have obtained sufficient data to properly characterise the rock mass and the stress field. The back analysis component is required in all cases in order to calibrate numerical model predictions and the damage and failure criteria to in situ observations of cracking. Closing the analysis with observations in this manner ensures a progression to appropriately dimensioned primary reinforcement.

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