Mine Health and Safety Council

MHSC

A handbook on practical design methodology for spans (bord widths) in bord and pillar mining on platinum and chrome mines in the Bush Veld Complex.

Project leader: S K Murphy

Project reviewer: W C Joughin, *Pr Eng*

Project team: E J Walls, *Pr Sci Nat*

Project team: C Zermatten, *Cand Sci Nat*

Research agency: SRK Consulting (Pty) Ltd (South Africa)

Project number: SIM 15-02-01

Date: March 2016

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List of abbreviations and symbols

°	Degrees
ε	Strain (millistrain)
σ	Stress (MPa)
2D	Two Dimensional
3D	Three Dimensional
BC	Bushveld Complex
CAD	Computer-Aided Design
DMR	Department of Mineral Resources
ESR	Excavation Support Ratio
FOG	Fall of Ground
GCD	Ground Control District
GPa	Gigapascal
JBlock	JBlock Software Program
kN	Kilonewton
kPa	Kilopascal
m	Metres
mm	Millimetres
m^3	Cubed Metres
MHSC	Mine Health and Safety Council
MPa	Megapascals
MRMR	Mining Rock Mass Rating
NGI	Norwegian Geotechnical Institute
RMR	Rock Mass Rating
RQD	Rock Quality Designation
UCS	Uniaxial Compressive Strength
SIM	Safety in Mines

SIMRAC	Safety in Mines Research Advisory Committee
SRK	SRK Consulting (Pty) Ltd (South Africa)
UDEC	Universal Distinct Element Code

Acknowledgements

The input data obtained from mines on both the Eastern and Western Limb of the Bushveld Complex, South Africa, formed the foundation of the research project. The data was vital in conducting an objective and comprehensive analytical review. Similarly, it facilitated a thorough understanding of the state and extent of current practices.

The Mine Health and Safety Council and SRK Consulting (Pty) Ltd (South Africa) extend their gratitude to the mines and personnel that contributed their technical documents, knowledge, insights and availability during interviews during the course of the project – namely: Bathopele, Glencore (Kroondal), Impala, Lanxess, Lonmin (Marikana) on the Western Limb and Booysendal North, Booysendal South (Everest), Glencore (Mototolo) and Two Rivers Platinum on the Eastern Limb.

Chapter 1 Introduction

Several major falls of ground have occurred in the past, at both platinum and chrome bord and pillar mines in the Bushveld Complex, South Africa. Several of these incidents have resulted in fatalities. The Mine Health and Safety Council, the Department of Mineral Resources and industry at large aim to transform the mining environment and achieve 'zero harm'.

In understanding the fall of ground (FoG) mechanism, the role of span width and pillar design are relevant. Pillar design methodologies are well-defined in published literature; in contrast, methodologies for the design of maximum stable spans (bord widths) are not as well-defined.

Initial considerations revealed that the most favoured approach for the design of the bord width appeared to be the use of an approach where the length of support is designed by specifying a support capacity requirement based on a 95% fall-out height for different ground conditions or ground control areas. This rule of thumb is taken from bord and pillar practices in the coal mining industry where the anchor support pattern, spacing layout, as well as the minimum drilling depth into the roof / hanging-wall, and thus the length of the anchors, is determined for a given mining area such that 95% of the potential fallout height is catered for. The rule is empirical / statistical in nature and requires a good historical sample of fall-out heights (i.e. the depth of the hole in the roof left after a fall of ground). The anchor support layout is usually on a square pattern at a spacing that is approximately 75% of the required support height. This engineering approach is often misused and misinterpreted and does not cater for the largest rockfalls. Other methodologies have been published (e.g. York *et al*., 1998 and Swart and Handley, 2005) but do not appear to be used.

The Mine Health and Safety Council (MHSC) initiated a research project to address the knowledge gap, in the form of a Safety in Mines (SIM) research project 15-02-01. SRK Consulting (Pty) Ltd (South Africa) (SRK) was successful in their proposal to the MHSC to lead the research project. SRK has conducted a great number of platinum group mineral (PGM) studies in recent years. Additionally, SRK also participated in the MHSC Project OTH501 into investigating factors governing the stability /instability of stope panels in order to define a suitable design methodology for near surface and shallow mining operations.

Subsequently, a research project was undertaken to develop methodologies and actionable knowledge required for a proper engineering approach applicable to the design of maximum spans (bord width) for platinum and chrome bord and pillar mines in the Bushveld Complex, South Africa. The project tasks included mine site visits; interviews; desktop review of all methodologies and technologies currently in use in hardrock bord and pillar mines in the Bushveld Complex, collaborative workshops and computational analyses.

It is envisioned that implementation of the outcomes of the research will improve health and safety in the South African Mining Industry by increasing the stability of the bord, resulting in a significant reduction in exposure to the ground conditions typically associated with rockfalls and collapses.

Chapter 2 Background

No universally adopted method exists for the design and / or evaluation of maximum span length of underground excavations, as affirmed by Swart and Handley (2005) in that '*very few mines design stope panels according to a systematic design procedure or methodology*'. This prompted the formulation of a design methodology for the design of conventional stope spans in the Bushveld Complex.

Stable roof spans in underground mining excavations are typically designed using empirically-based techniques (Swart, 2005 and Esterhuizen *et al*., 2011) and these may be supplemented by analytical (mathematical) methods. It appears that modified approaches, combination of approaches or hybridised versions of different approaches for different excavation environments (e.g. tunnelling, stoping etc.) are utilised in designing the span for bord and pillar mining. The most common design approaches are listed as follows:

- Empirical design methods using:
    - Observations and / or past experience
    - Rock mass classification systems
    - Stability graphs
- Analytical [computational] design and / or validation methods using:
    - Numerical modelling
    - Beam analysis

Although variations of the design approaches identified above are being implemented (albeit haphazardly) at different operations, 'a spate of fall-of-ground accidents' in the bord and pillar mining areas of platinum / chrome sector of South Africa's mining industry has occurred (Singh *et al*., 2010). The accidents prompted the MHSC to assemble a team of experts from academia and industry during the year 2010, with the purpose of investigating the large collapses, the cause of the FoG incidents and subsequently to identify mechanisms for the prevention of further FoG incidents (where large collapses are considered those measuring larger than either 10.0 m^2 in area or 5.0 m^3 in volume) (Singh *et al*., 2010).

The panel of experts considered various aspects that may influence the stability of the hanging-wall in platinum mines (irrespective of the type of mine – conventional, mechanised or combination thereof). The panel concluded, amongst other information, that:

1. There is an incomplete understanding of the characteristics of the Bushveld Complex.
2. Adverse geology, as well as '*low angled, dome-like joints' 'between 30°-60°'* (Singh *et al.,* 2010*)* contributed significantly to the large FoG incidents.
3. Preliminary analysis of available accident statistics revealed that unfavourable structures in the rock mass and / or unfavourable situations were not identified correctly and / or were not treated correctly.
4. Issues regarding skills and capacity of mining and rock engineering personnel were also an important factor during unsafe operations.

The findings reported by the panel of experts are relevant in that they set out the context, challenges and key structures (i.e. low angled joints) that need to be addressed with additional care and attention, during the design of maximum stable bord widths.

Several authors have also commented and / or criticised that desired roof span dimensions or underground excavations are largely predetermined by equipment and operational requirements. The consequent design is intended to ensure stability of these required geometries under prevailing rock conditions (e.g. Swart, 2005; Hoek, 2008 and Esterhuizen *et al.,* 2011).

In 2013, a Bord and Pillar Task Team was established by the Department of Mineral Resources (DMR) Principal Inspector of the North West Region, to determine current industry best practice for bord and pillar mining (Platinum Task Team, 2013). It was found that there was a variation in the methodologies used to design bord widths but the majority of operations used the Voussoir beam theory to determine stable spans.

It is imperative that an appropriate engineering approach / design methodology is utilised to design stable spans and identify support requirements. Furthermore, the development of a proper engineering approach for the design of maximum stable bord widths should be holistic and consider all types of failure mechanisms that may occur, i.e. intact rock mass, structural, beam and rock mass failure. The methodology therefore needs to represent a synergy of the existing elements employed by various mines selectively in parts and / or at different stages of the design process.

Chapter 3 Methodology for the design of maximum stable bord width

The approach to the design and evaluation of underground mining excavations, with regards to stability, follows a straight-forward path. Stacey (2001) describes the journey and considers important aspects, such as the purpose of the excavation and the stability of the excavation in relation to the quality of the rock mass.

The purpose of the excavation determines its geometry and size, for example:

- The mining extraction excavation geometry is dictated by the orebody shape and, within the context of this book, bord and pillar mining method is the chosen method.
- The practicality and stability of the excavation must then be evaluated in relation to the quality of the rock mass in which it is located.
- Is it, or will it be, stable?
- What is the mode of identified instability, if any?
- Can the instability be overcome by modifying the geometry and location of the excavation?
- What support, if any (quantity and type), is necessary to ensure that the desired stability is achieved?

In the context of bord and pillar mining method, the attributes regarding the purpose of the excavation are well-decided. In contrast, the practicality and stability of the excavation require confirmation and have been used to guide the design approach in the current research project for determining maximum bord width.

3.1. Modes of instability

Haile and Jager (1995) identified six different modes of failure in pillar-supported hard rock mines within the Bushveld Complex. An additional two were identified during this project. These modes are categorised below into four types of failure.

Rock Mass Failure

- ***Unravelling failure***: Occurs when the hanging-wall of the stope contains a prominent joint set of uniform dip and dip direction and the hanging-wall span between pillars exceeds a certain critical limit.

Beam Failure

- ***Buckling failure***: When the hanging-wall beam buckles and failure is not defined solely by joint geometry.
- ***Beam shear failure***: Failure occurs due to slip on widely spaced and subvertical planes of weakness or initiated as fractures close to pillars or abutments.

Structural Failure

- ***Keyblock failure***: Where two or more mutually intersecting joints are present in the bord hanging-wall and create unstable block geometry.
- ***Wedge failure***: Where two major planes of weakness intersect in the stope hanging-wall. The areal extent of the failure is generally far greater than that of keyblock failure.
- ***Cooling dome / low-angled joints / ramp fault failure***: Failure is initiated due to fallout on shallow-dipping structures on the periphery of a convex (upward-curving) basin shaped block of rock. Domes are approximately circular in shape and vary in size from a few square metres to several hundred square metres. They are common across the whole of the Bushveld Complex.

Intact rock failure

- ***Tensile failure*****:** This type of failure is driven by a potential tensile dome existing in the hanging-wall of a stope. This may occur at shallow depths in a bord and pillar environment.
- ***High horizontal stress:*** Failure associated with high horizontal stress has been identified at shallow to intermediate depths. (Esterhuizen *et al.,* 2011, Watson 2003).

As introduced in Chapter 2, the development of a proper engineering approach for the design of maximum stable bord widths should be holistic and consider all types of failure mechanisms that may occur i.e. intact rock mass, structural, beam and rock mass failures. Swart and Handley (2005) developed a methodology for stope panel spans in shallow mining operations, which was adapted for use with the design of stable bord widths in the research project SIM 15-02-01. The methodology identifies the inputs required, as well as, the analysis to be carried out and is illustrated in Figure 1. Each of the analyses, linked to a failure mechanism, is described in further detail in the paragraphs that follow. It is important to note that the approach is to be tailored for the specific mining operation especially where low-angled joints are present in the rock mass of the specific site.

The methodology has a reiterative component namely: performance of the rock engineering environment must be continually monitored, managed and included in the development of the design. Additionally, as the geotechnical database is expanded, the new information must be used to update and inform the design process.

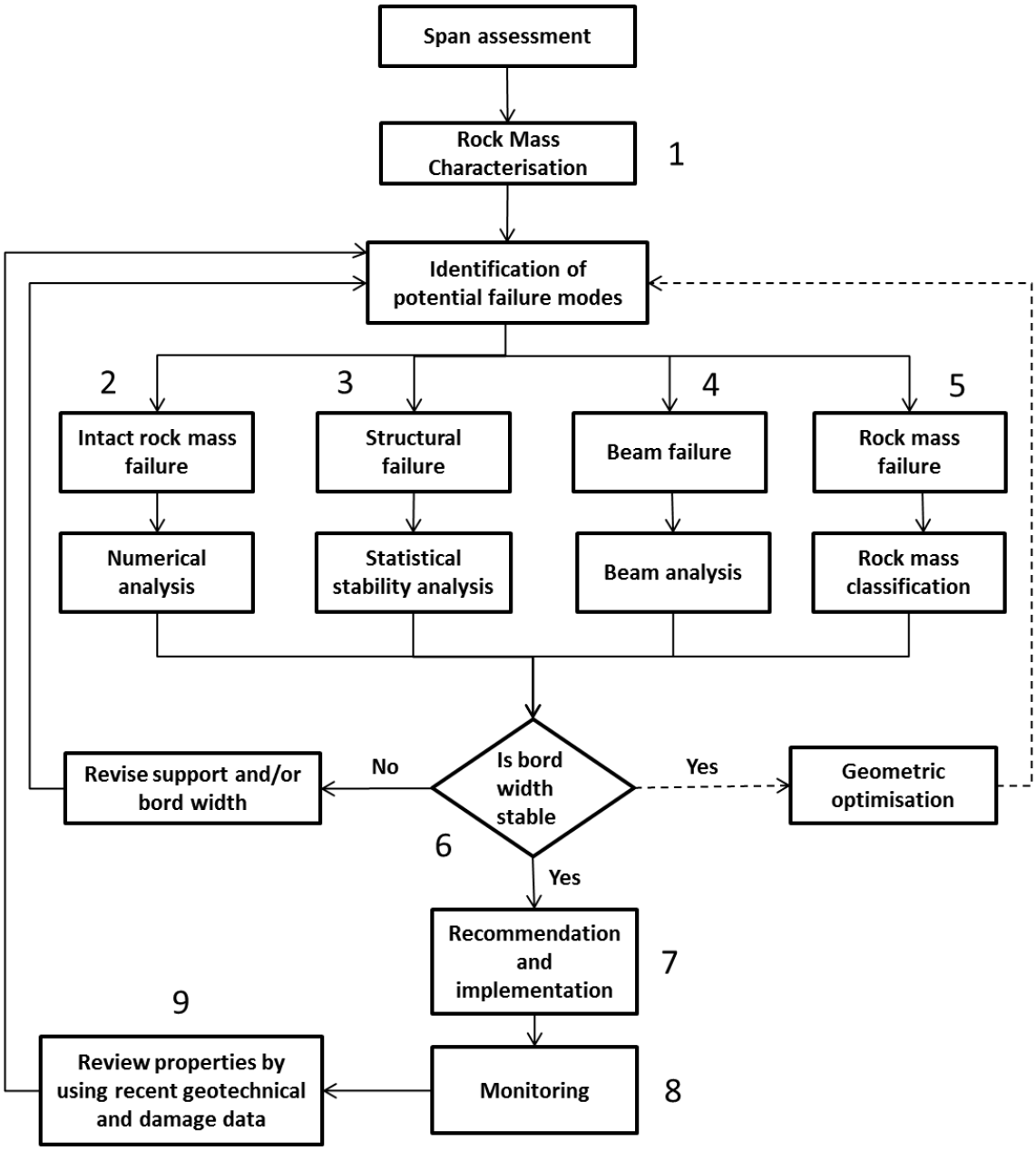

Figure 1: **Methodology for the design of maximum stable bord width (Modified from Swart and Handley, 2005)**

3.2. Rock mass characterisation (1)

The design process for maximum stable bord width, as for pillar design, is reliant on technical information from the rock mass database. To this end, a rock mass database must be compiled, made available for use and updated as new information becomes available. The database must record all geotechnical aspects relevant to ground support and stability assessments, including:

1. Rock type
2. Rock strength
3. Discontinuity type
4. Discontinuity persistence (dip and strike)
5. Discontinuity orientation relative to the excavation
6. Spacing between and trace length of discontinuities
7. Discontinuity characteristics (profile, thickness of infill, type of infill)
8. Weatherability of the intact rock

There is no prescription on how the information must be collected. Common techniques include mapping and borehole core logging. The use of cameras in boreholes is increasingly useful in identifying low-angled joints (where professional experience and discussion, as well as Singh *et al*., 2010 reveal that low-angled joints are those joints 0°- 60°, measured from the horizontal).

3.3. Rock mass failure (2)

Background

Swart *et al.* (2000) identified four rock mass classification systems for evaluating the stability of stope spans in the Bushveld Complex, namely:

1. The Geomechanics classification or Rock Mass Rating (RMR) system developed by Bieniawski in 1973.
2. The Norwegian Geotechnical Institute (NGI) rock quality index or Q-system developed by Barton *et al.* in 1974.
3. The Modified Rock Mass Rating (MRMR) system originally developed by Laubscher in 1974.
4. The modified stability graph method through the use of the modified stability number, N', originally developed by Mathews *et al*. in 1980.

An empirical assessment based on practical experience that caters for a variety of mechanisms is useful to address the potential for unravelling failure, which is not catered for in mechanistic approaches. The poorest rock quality (as per the rock mass classification) can be expected to unravel.

Esterhuizen *et al.* (2011) notes that empirical methods based on rock mass classification are used extensively to obtain an initial indication of 'likely' dimensions of stable spans under given rock mass conditions, such as Bieniawski (1989); Barton *et al.* (1974); Laubscher (1990) and Mathews *et al.* (1980). Such classifications are also used to produce an estimate of the span and corresponding support requirements.

On the basis of an evaluation of a large number of case histories of underground civil engineering excavations (most of which were supported) Barton *et al.* (1974) of the NGI, proposed the Q-system rock mass classification for the determination of rock mass characteristics and tunnel support requirements. Q can be calculated using Equation 1:

$$Q = RQD/Jn \times Jr/Ja \times Jw/SRF \qquad \text{Equation 1}$$

Where:

RQD Rock Quality Designation

J_n Joint set number

J_r Joint roughness number

J_a Joint alteration number

J_w Joint reduction number

SRF Stress Reduction Factor

Barton *et al.* (1974) show that the maximum unsupported span can be estimated using Equation 2:

$$\text{Maximum Span Maximum Span (unsupported)} = 2\ \text{ESR}\ Q^{0.4} \quad \text{Equation 2}$$

Where:

ESR (Excavation Support Ratio) is a value that is assigned to an excavation in terms of the degree of security that is demanded of the installed support system to maintain the stability of the excavation (Barton *et al.,* 1974).

The modified rock quality index (Q') (Mathews *et al.,* 1980) can be calculated using Equation 3:

$$Q = RQD/Jn \times Jr/Ja \quad \text{Equation 3}$$

The general Matthews stability chart is then used to estimate stability for unsupported spans. Potvin (1988) and Nickson (1992) studied supported cases where the spans had been stabilised with cable bolts (Hutchinson and Diederichs, 1996). The modified stability number (N') specified by Potvin (1988) is calculated using Equation 4:

$$N' = Q' \times A \times B \times C \quad \text{Equation 4}$$

Where:

A Is the ratio of intact rock strength to induced stress.

B Is a measure of the relative orientation of dominant jointing to the excavation surface.

C Is the influence of gravity on the stability of the face being considered.

Application

This method of assessment is suited to a jointed rock mass when the hanging-wall of the bord contains one or more prominent joint sets of uniform dip and dip direction and / or cooling dome / low-angled joints / ramp faults. Then if the hanging-wall span between pillars exceeds a certain critical limit unravelling will occur.

For example, Merensky reef rock mass data was extracted from York *et al*. (1998) according to Impala Mines' adaptation of the '*NGI Tunnelling Quality Index'*, Table 1.

Table 1: Merensky reef rock mass data (after York *et al*. 1998)

Parameter	Union-1 rating	Amandel-1 rating	Lebowa-1 rating	Amandel-2 rating	Amandel-3 rating	Amandel-4 rating	Lebowa-2 rating
RQD	44	57	82	100	89	98	100
Jn	6	6	12	6	12	6	12
Jr	0.5	1	1	1	1	1	1.5
Ja	8	4	4	2.5	4	4	4
Q'	0.46	2.38	1.71	6.67	1.85	4.08	3.13

In the room and pillar environment, a systematic support system is used to primarily provide reinforcement to the immediate hanging-wall and to protect against small discontinuities and blasting damage. This is usually done using tendons such as resin or cement grouted bolts or swellex type bolts. In the bord and pillar environment this can be considered to be similar to the stoping environment support. Therefore within the context of this example the bord is always supported with some form of tendon support. Q' was calculated for the data, using Equation 3.

In order to assess stability using a modified stability chart the Hydraulic Radius (HR) of the bord width is calculated using Equation 5. However, in the case of a bord with an infinite length (as with a bord and pillar layout) the HR will be approximately half the width of the bord.

$$HR = \text{Area } (m^2) / \text{Perimeter } (m) \qquad \text{Equation 5}$$

Next, the modified stability number N' is calculated using Equation 4, where A B and C are obtained from the charts shown in Figure 2 to Figure 4.

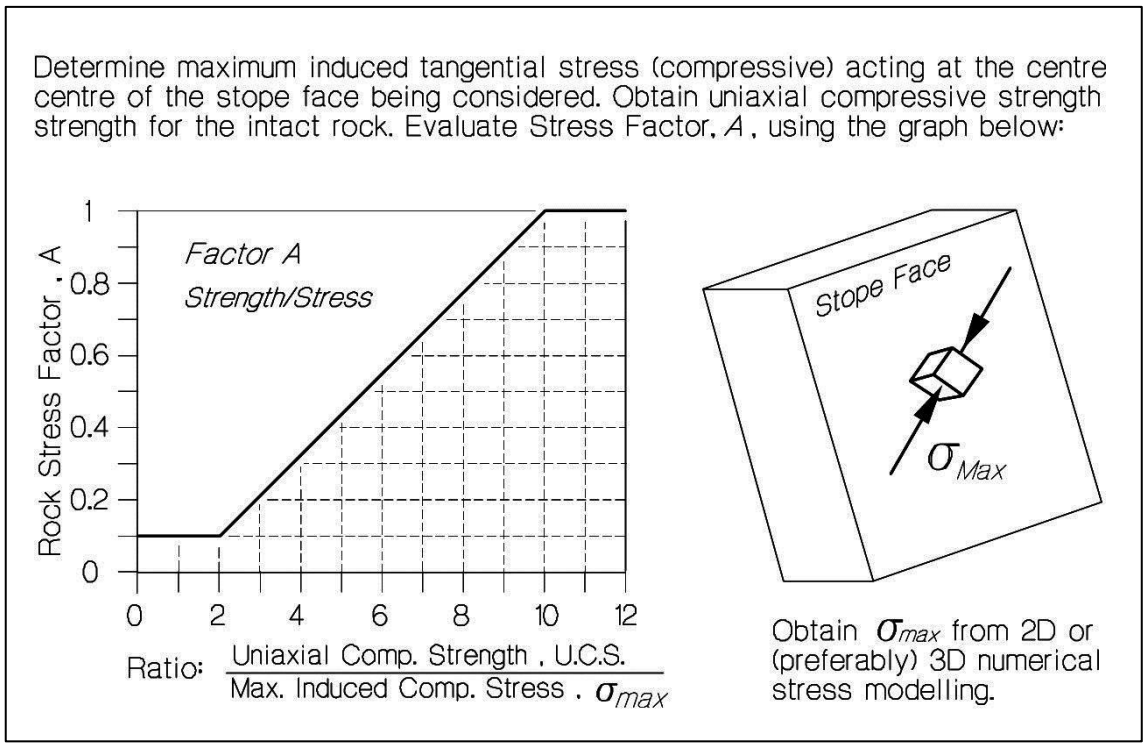

Figure 2: Rock stress factor (Hutchinson and Diedrichs, 1996)

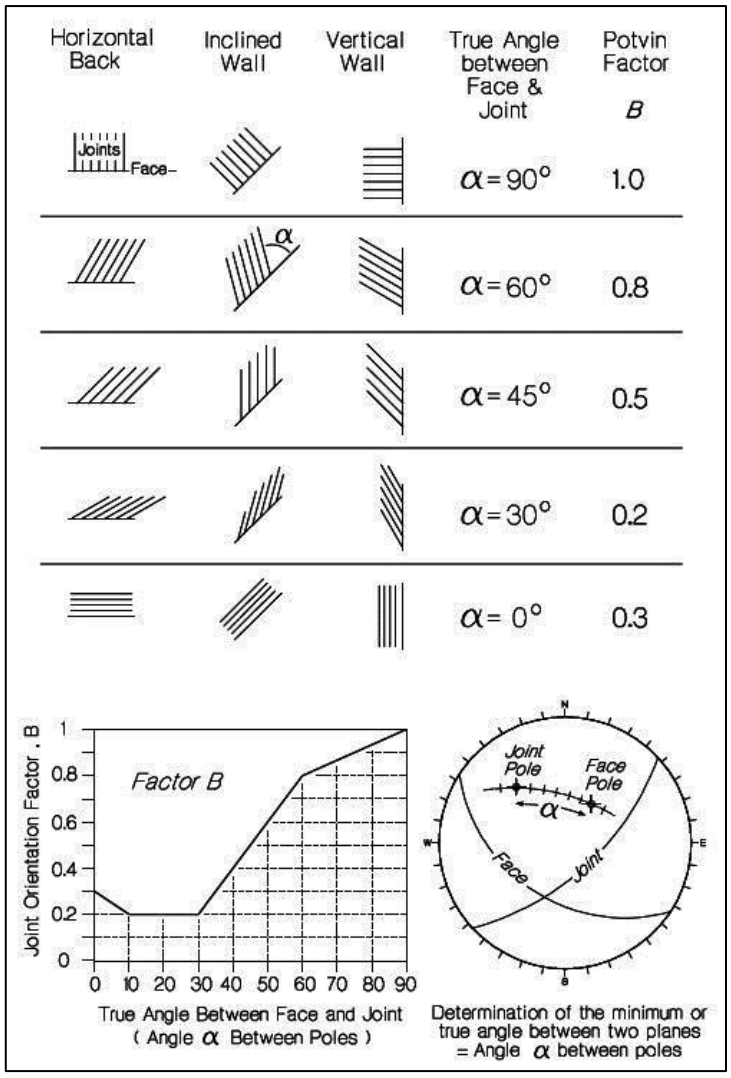

Figure 3: Joint orientation factor B (Hutchinson and Diedrichs, 1996)

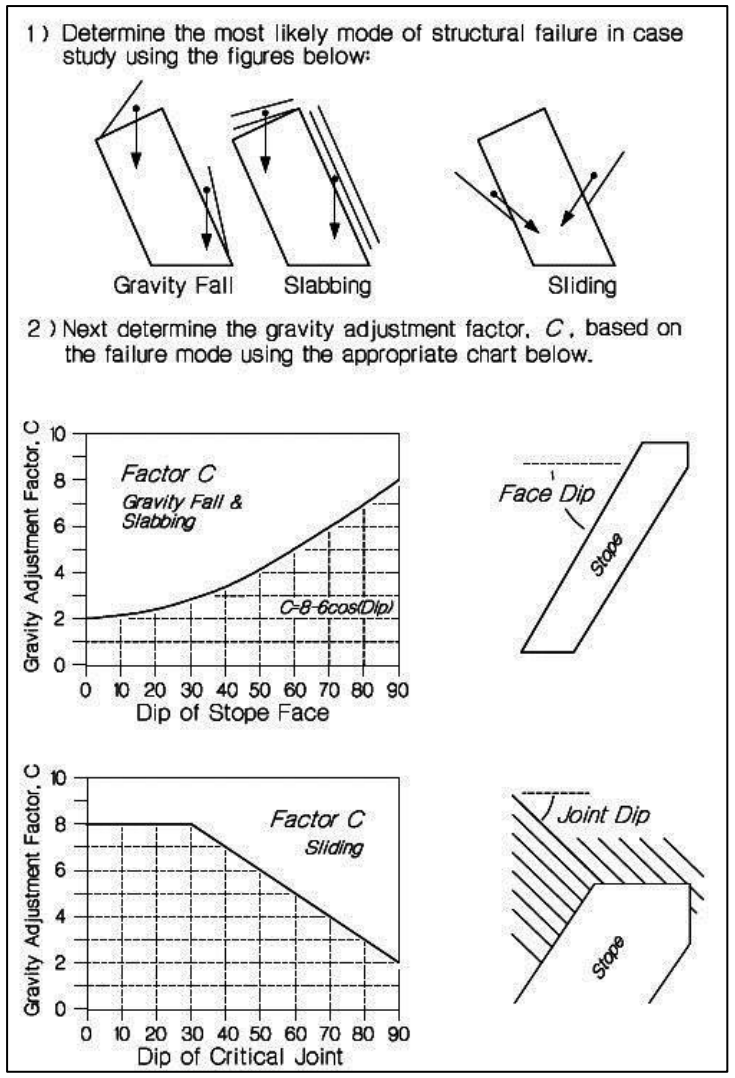

Figure 4: Gravity adjustment factor C (Hutchinson and Diedrichs, 1996)

For the rock stress factor (A), where the k-ratio (σ_h / σ_v) is approximately 1, and a depth of 500 m, it can be assumed that for an almost horizontal excavation, where the intact rock strength is in excess of 200MPa, the A value will be 1.
In this example, two cases were chosen from Table 1, namely Union-1 as the worst case and Amandel-4 as the best case. For the dip of the most critical joint at Union1, a low-angled joint of 40° was chosen and for Amandel-4, the critical joint was chosen to be 70°. In bord and pillar mining where the dip is usually less than 10°, and using Figure 3, the B value for Union-1 is 0.2 and for Amandel-4 is 0.8.

The gravity adjustment factor C for the hanging-wall with a dip of 10° is 2.

Results

Results are presented in Figure 5 and Figure 6 where it can be seen that for Amandel-1 the standard support system is sufficient to cater for stability; however at a 12 m span the probability of failure is 5%. In contrast, for Union-1 for the span between 6 m and 12 m will require additional cable bolting / long anchors.

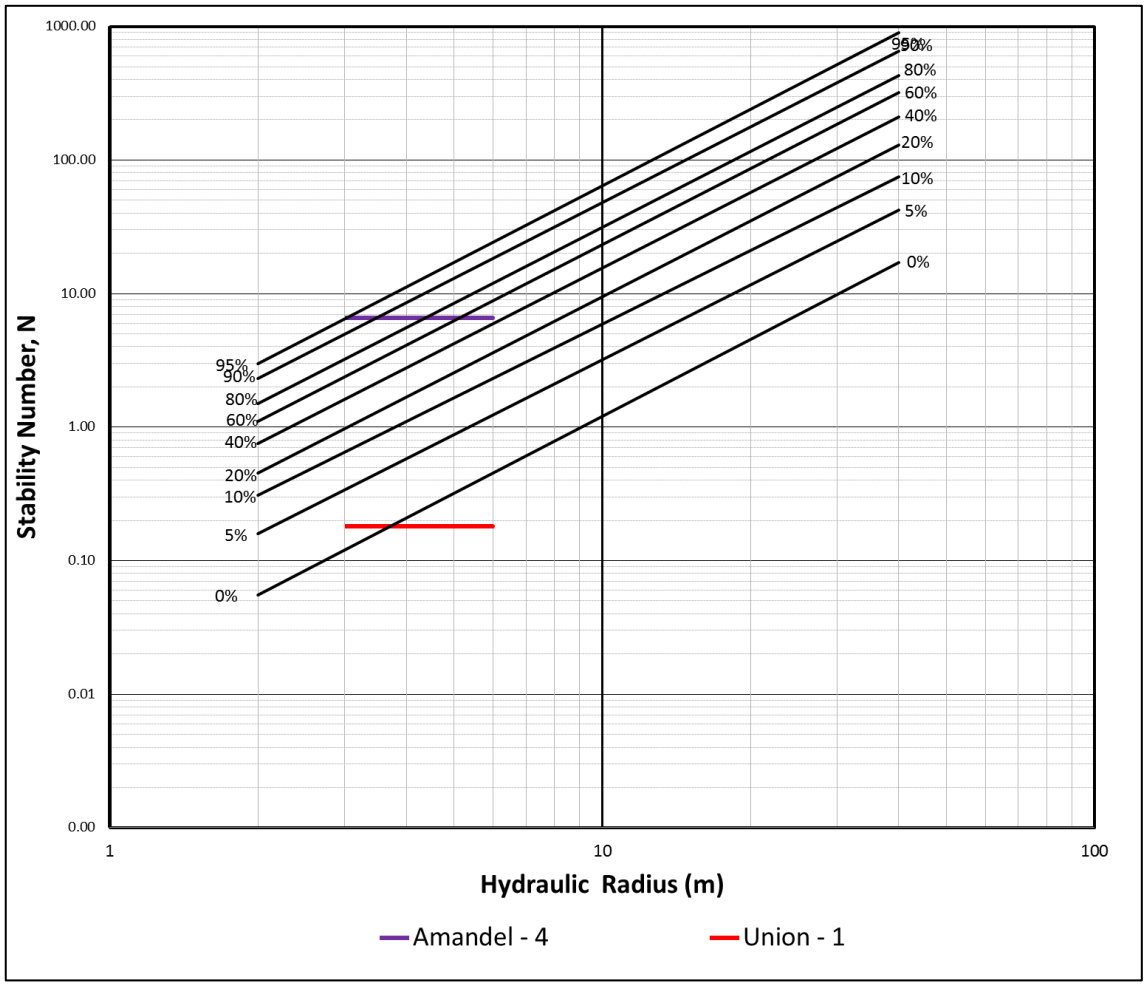

Figure 5: Matthews Method Risk-based stability graph (after Mawdesley 2003).

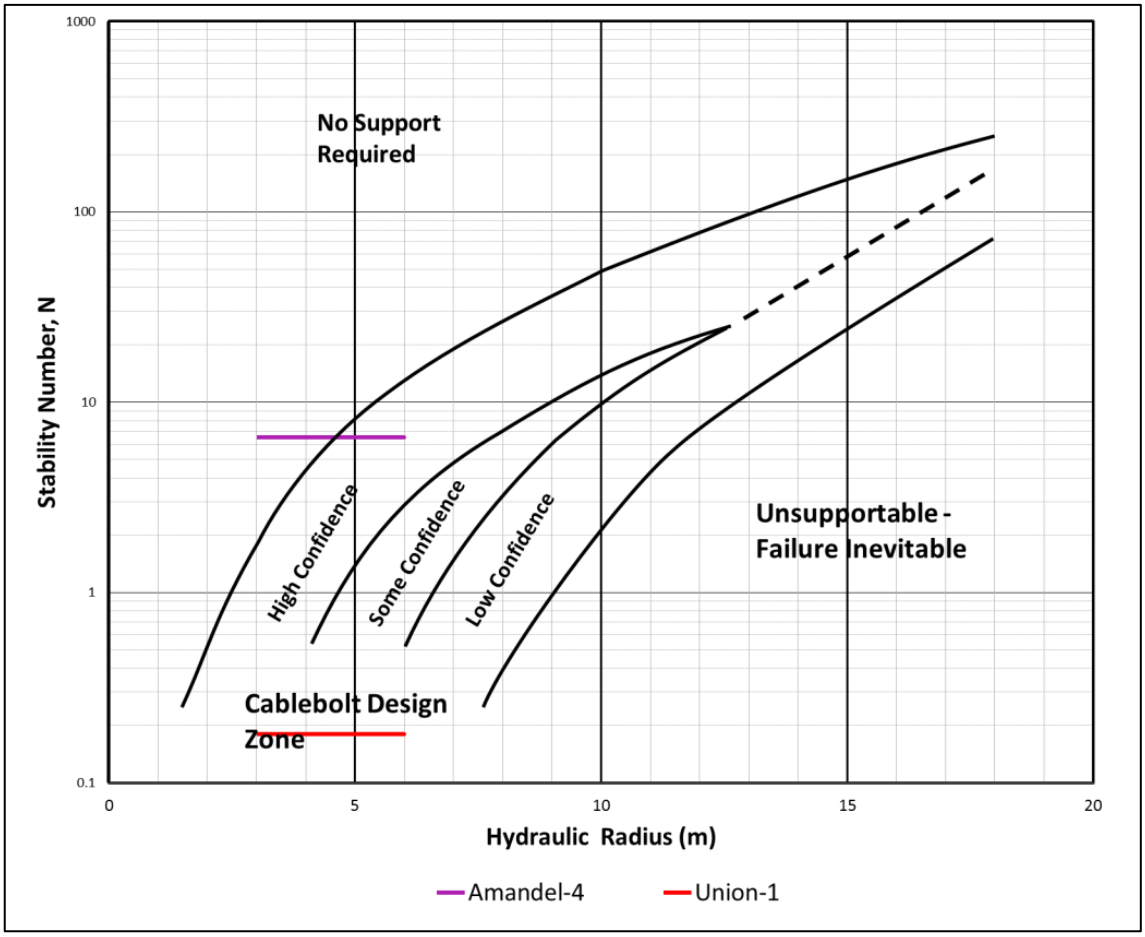

Figure 6: Potvin (1988) Method developed for cable bolt support design (after Diedrichs and Hutchinson 1996)

Conclusion

This methodology appears to identify the problems that are associated with flatdipping joints and the potential for unravelling. The unravelling that occurred at Union-1 site resulted in a measured closure of 340 mm when 35 mm elastic closure occurred (York *et al*., 1998).

It also indicated that when low-angled joints are encountered the probability of failure is extremely high in whatever bord width is being used and cable anchors / long anchors will be required. The length of these cable bolts would be dependent on the bord width and the angle of the flat-dipping discontinuity. Assuming the flat-dipping discontinuity (say 60°) intersects the bord adjacent to the pillar then for a 12 m bord the length of the cable anchor would need to be in excess of 6 m to adequately anchor the support above the height of the potential fall.

3.4. Beam Failure (3)

Background

Elastic beam theory is useful in explaining the deformation and failure of a bord in stratified and pseudo-stratified deposits. However, in the Bushveld Complex, the existence of sub-vertical jointing is commonplace, which results in the tensile strength of a rock beam being zero. A stable rock beam in a bord will occur if a compressive arch (Voussoir arch) can develop. Notably, the Codes of Practice of several of the mining operations visited during the research project mention that that a Voussoir arch methodology was being used.

Diederichs and Kaiser (1999) describe an iterative solution system to determine the stability and the deflection of a Voussoir arch by calculating the buckling limit, the factor of safety for the crushing at midspan and at the abutments, the factor of safety for sliding at the abutments, and the midspan deflection. A buckling parameter of 35% has been determined (Hutchinson and Diederichs, 1996) as a limit above which a beam should be considered as unstable. This design limit corresponds to a midspan deflection of 10% of the beam thickness. Therefore, in a Voussoir arch stability analysis when the mid span displacements reach approximately 10% of the beam thickness, arch collapse is imminent. Additionally, arch stability can be assessed by monitoring the displacement at mid-span (Swart *et al.*, 2000).

Application

When considering the strength of the hanging-wall in the Bushveld Complex and using the chart shown in Figure 7, it can be seen that for a beam thickness of 0.5 m, a 20 m span would be stable.

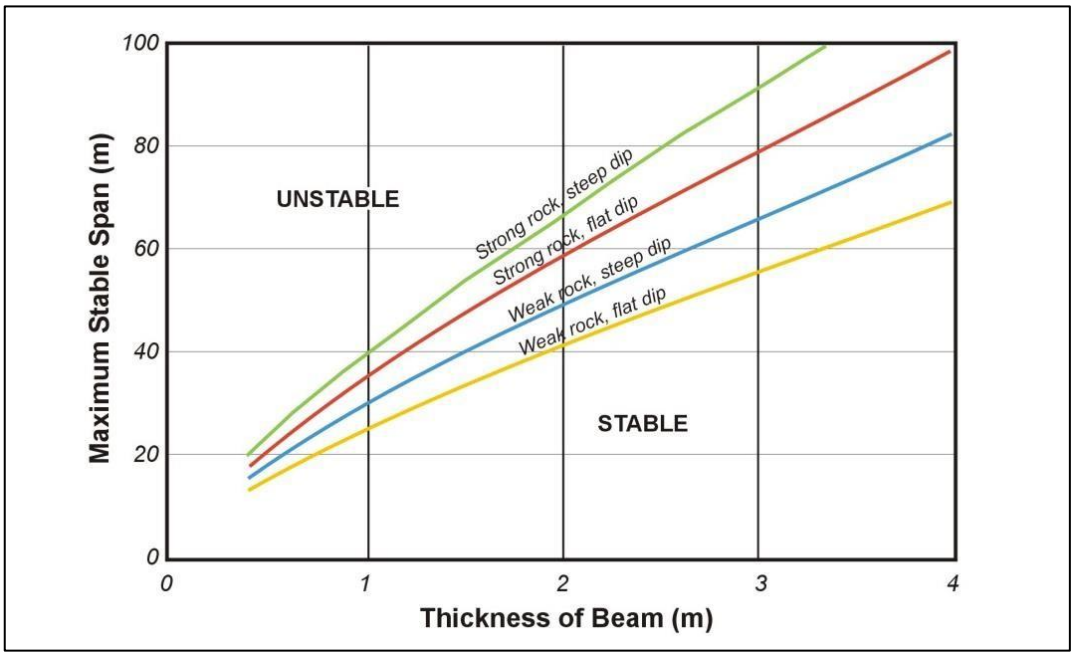

Figure 7: Stability of beams along excavation (Stacey, 2001)

It must be noted that this methodology is only applicable to areas where there are no flat-dipping joints (0° to 60°). Also potential wedges formed by joints in the beam and weathering will prevent the rock beam performing as an arch.

Conclusion

Beam failure is an important assessment but typical bord and pillar layouts will “pass” such a test, as demonstrated in the example above. It is only in locally poor ground conditions that the span may fail the Voussoir arch stability test.

3.5. Structural failure (4)

Background

Structural failures that can be assessed using statistical stability analysis are keyblock, wedge and “cooling dome” / low-angled joints / ramp fault failure. It has been observed that many of the large rockfalls that occur in bord and pillar layouts are joint bound. Natural joints in the rock mass form keyblocks that vary in size from very small blocks to extremely large blocks depending on how the joints interact to form the block. Some of the blocks will be naturally stable, while others will be unstable due to unfavourable joint orientations and / or low joint shear strength and will require tendon support and pillars to keep them in place. The JBlock software program has been used to identify keyblocks and probabilistic failure potential (Esterhuizen, 1996). Probabilistic risk evaluation for support design, model validation and case studies by Joughin *et al*. (2012a, 2011b) and Walls *et al*. (2013) provide a method utilising JBlock and RiskEval as part of a programme for risk based support design in underground mines in the Bushveld Complex.

Application

The software program JBlock is capable of simulating the formation of many thousands of blocks and testing the stability of each block. Block formation is based on mapped joint data for a given ground control district (GCD). Typical ground conditions for a particular GCD are therefore represented. JBlock allows for the span between pillars, as well as the type and length of bolts to be designed in such a way as to minimise the risk of rockfalls and by association, minimise injuries. Furthermore, JBlock can simulate mining layouts, different support patterns and types of support. The risk of large rockfalls can then be assessed on a comparative basis.

Results

By way of example, two GCDs (GCD1 and GCD2) were simulated. The joint characteristics are summarised in Table 2 and Table 3. GCD1 has only three joints sets, but includes a parting plane with a mean distance of 0.9 m above the hangingwall contact and a standard deviation of 0.2 m. GCD2 has four joints, but the fourth joint set is flat-dipping (20°). Note that the maximum trace lengths in GCD1 are much larger than in GCD2.

Two mining layouts were compared, each with a chequerboard pillar pattern, one with a 12 m bord width and the other with a 6 m bord width. In both cases resin bolts were simulated in 1.5 m x 1.5 m pattern. A tensile strength of 20 kN and a bond shear strength of 400 kN/m was used. For the 6 m bord width, 1.5 m long resin bolts were simulated and for the 12 m bord width, a simulation was carried out with 1.5 m bolts and another with 3.0 m bolts. The objective of the analysis was to compare spans and bolt lengths.

The results of the statistical key-block analyses are presented in Table 4 and Figure 8. It is apparent that reducing the bord does mitigate the rockfall risk by reducing the percentage of blocks that fail and the rockfall sizes. Increasing the length of the bolts is equally effective in GCD1, but not in GCD2.

Table 2: GCD1 Joint characteristics

	Orientation				Strength				Spacing (m)			Length (m)		
	Dip (°)		Dip Direction (°)		Cohesion (kPa)		Friction angle (°)		Mean	Maximum	Minimum	Mean	Maximum	Minimum
Joint sets	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation						
Set1	80.0	5.0	202.0	13.0	0.0	0.0	33.0	5.0	1.3	9.1	0.5	10.0	10.0	6.0
Set2	82.0	4.0	279.0	10.0	0.0	0.0	33.0	5.0	0.7	3.9	0.5	5.0	10.0	0.5
Set3	83.0	4.0	240.0	86.0	0.0	0.0	33.0	5.0	2.6	9.9	0.4	5.0	10.0	0.5

Table 3: GCD2 Joint characteristics

	Orientation				Strength				Spacing (m)			Length (m)		
	Dip (°)		Dip Direction (°)		Cohesion (kPa)		Friction angle (°)		Mean	Maximum	Minimum	Mean	Maximum	Minimum
Joint sets	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation						
Set1	66.0	4.8	356.0	2.4	0.0	0.0	31.0	18.0	0.7	4.4	0.5	2.7	2.8	2.5
Set2	73.0	4.7	175.0	4.6	0.0	0.0	35.0	7.0	0.9	5.3	0.5	3.1	10.0	1.8
Set3	74.0	5.3	266.0	10.1	0.0	0.0	34.0	16.5	1.3	5.6	0.5	2.4	3.6	1.8
Set4	20.0	1.0	278.0	1.0	0.0	0.0	33.0	1.0	10.0	30.0	0.5	2.4	2.8	2.1

Table 4: Summary results of statistical keyblock analysis

	Failed Blocks (%)	Failed Block Area (%)	Maximum Volume (m^3)	Maximum Area (m^2)	Maximum Height (m)
GCD1 Span 12m Tendons 3m	0.06	0.08	8.8	9.3	1.4
GCD1 Span 12m Tendons 1.5m	0.08	0.12	19.9	13.5	2.9
GCD1 Span 6m Tendons 1.5m	0.05	0.08	13.1	10.2	1.3
GCD2 Span 12m Tendons 3m	1.69	1.86	5.1	5.5	2.3
GCD2 Span 12m Tendons 1.5m	1.79	2.07	5.1	5.0	2.3
GCD2 Span 6m Tendon 1.5m	0.91	0.92	4.8	5.0	2.3

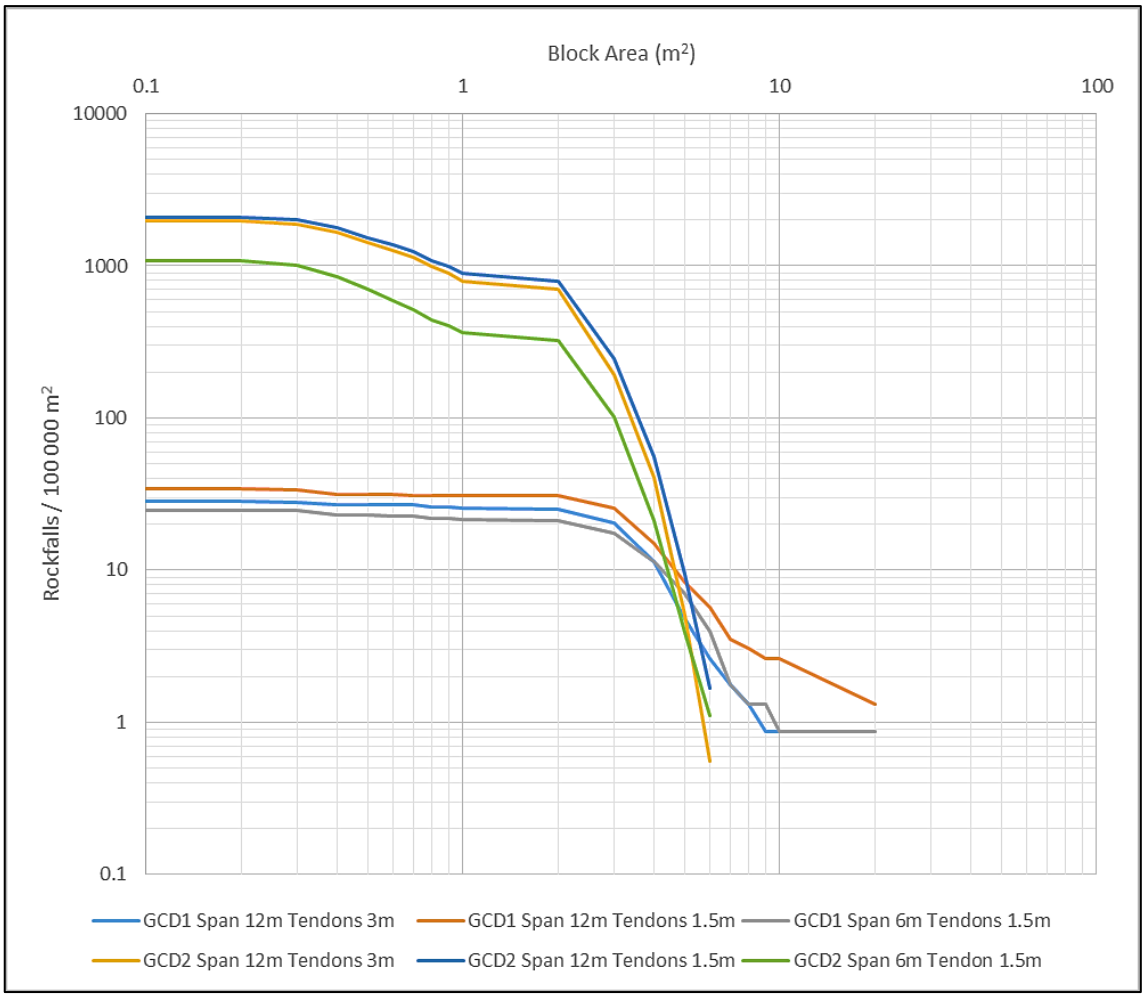

Figure 8: Rockfall area cumulative frequency distribution (rockfalls are normalised per 100 000 m^2 mined).

Conclusion

The two different GCDs produce very different results. Note that the maximum rockfall size is much greater in GCD1 than in GCD2 - this is due to the large trace lengths in GCD1. Conversely, the rockfall frequency in GCD2 is much greater due to the number of joint sets and the flat-dipping joints. Many very small rockfalls (<1.0 m^3) occur in GCD2, while few small rockfalls occur in GCD1. In JBlock, the larger rockfalls invariably fail due to block rotation, while the small rockfalls fall out inbetween support. This is representative of the actual situation underground.

It should be noted that reducing the span does not eliminate the occurrence of large or small rockfalls and additional measures will therefore be required to manage the risk. The frequency of large rockfalls can be further reduced by introducing longer and stronger bolts, such as 38 tonne cable anchors. However, since these large rockfalls are infrequent, it will invariably be more cost effective to implement an effective monitoring programme (e.g. borehole camera or ground penetrating radar) and only install the additional support when it is required. The small rockfalls in GCD2 can be mitigated by barring or introducing areal support such as mesh.

3.6. Intact rock failure (5)

Background

It has been suggested (Nyungu and Stacey, 2014) '*that large zones of tensile strain may occur around Bushveld Complex (BC) excavations where the magnitude of the extension strain exceeds the critical value determined during laboratory testing. The predicted orientations of these models correspond with observed geometry of spalling in excavations. The implication is that there are likely to be substantial zones surrounding BC mine excavations that may be prone to spalling conditions and perhaps more significant failure'*. This coupled with extension fractures observed at varying depths by Watson (2003) in the BC and potential extension stress failure observed at a major fall of ground in a bord and pillar mine at a depth of 1 200 m indicates that as the bord and pillar mining in the Bushveld progress deeper extension failure may need to be included as part of bord stability design.

Therefore, a numerical assessment is useful in assessing the intact rock failure associated with tensile stress or high horizontal stress under modes of instability. The software program Map3D allows for visualisation of three-dimensional models, using a built-in Computer-Aided Design (CAD) functionality (Map3D, 2015). A methodology described by Stacey (1981) was used to assess the potential of extension strain failure initiation in brittle rock which is particularly applicable in areas of low confining stress. These areas occur around underground excavations and this criterion may be suitable for the prediction of the extent of fracturing around the excavation and thus provide input for both support design and the design of appropriate maximum spans in bord and pillar mines.

Application

This type of modelling will be useful to determine:

- The height of the tensile zone.
- Stress failure in areas where high horizontal stresses can be expected (around potholes etc.).
- The potential height of failure that may be expected as the depth of the bord and pillar mining increases.

Numerical modelling and criterion

In considering Map3D numerical modelling using an extension strain criterion, the input parameters for the Bushveld Complex rocks were obtained from a published paper detailing both uniaxial compressive strength (UCS) test results and Brazilian indirect tensile tests (Nyungu 2013; Nyungu and Stacey, 2014), shown Table 5 and Table 6.

Table 5: Summary of average results from UCS tests

Rock type	Mottled Anorthosite	Spotted anorthositic norite	Pyroxenite	Mottled anorthosite	Norite	Spotted anorthositic norite	Anorthositic norite	Spotted anorthosite	Mottled anorthosite
Code	**(A)**	**(B)**	**(C)**	**(D)**	**(E)**	**(F)**	**(G)**	**(H)**	**(I)**
Sample diameter, D (mm)	36.30	36.30	36.30	36.30	36.30	36.30	36.30	36.30	36.30
Sample length, L (mm)	80.74	84.79	81.66	81.13	83.71	82.87	80.99	81.03	80.98
L/D ratio	2.23	2.34	2.25	2.24	2.29	2.28	2.23	2.23	2.23
Sample mass, M (g)	231.41	254.20	270.30	230.93	261.32	248.10	253.48	237.80	232.32
Sample density, ρ (kg/m^3)	2769.46	2898.71	3198.41	2750.49	3016.40	2892.39	2990.22	2832.12	2772.17
Failure load, (kN)	180.60	139.40	129.80	140.50	96.00	154.60	114.00	159.60	182.20
UCS, σ_c (MPa)	744.51	134.70	125.42	135.76	92.76	149.38	110.15	154.22	176.05
Elastic Modulus, E (GPa)	44.60	33.32	35.49	39.01	30.90	40.65	37.90	42.64	45.31
Poisson's ratio, *v*	0.20	0.21	0.17	0.28	0.19	0.21	0.15	0.22.	0.19
Long term strength (MPa)	90.20	61.80	56.50	59.75	53.50	75.60	83.60	103.33	125.75
% of UCS	57.00	46.00	44.00	44.00	57.00	51.40	72.40	67.00	68.75

Table 6: Summary of Brazilian indirect tensile tests

Rock type	Sample ID	Sample diameter, D (mm)	Sample thickness, t (mm)	t/D ratio	Sample mass, M (g)	Average load at failure P (kN)	Average BIT strength α (MPa)	Average elastic Modulus, E (GPa)	Average Strain at failure (millistrain)	Average time-to-failure (s)
Mottled Anorthosite (A)	**DBA**	36.30	19.27	0.53	54.26	8.19	7.46	46.72	0.16	220.71
Spotted anorthositic norite (B)	**DBB**	36.30	18.88	0.52	55.46	6.84	6.35	33.32	0.19	205.72
Pyroxenite (C)	**DBC**	36.30	18.97	0.52	62.67	7.43	6.89	35.4	0.19	206.77
Mottled anorthosite (D)	**DBD**	36.30	18.77	0.52	53.84	6.83	6.38	39.01	0.16	138.02
Norite (E)	**DBE**	36.30	18.36	0.51	57.67	6.92	6.62	30.9	0.21	160.35
Spotted anorthositic norite (F)	**DBF**	36.30	18.61	0.51	55.24	8.27	7.76	40.65	0.19	138.17
Anorthositic norite (G)	**DBG**	36.30	17.65	0.49	56.31	7.71	7.65	37.9	0.2	203.85
Spotted anorthosite (H)	**DBH**	36.30	17.59	0.48	51.29	7.11	7.1	42.64	0.17	213.83
Mottled anorthosite (I)	**DBI**	36.30	17.06	0.47	48.93	6.82	7.04	45.31	0.16	214.03

The numerical models for this exercise were built in Map3D using fictitious force elements to represent the bords. Numerical models were run for a bord width of 6 m and a pillar width of 6 m (Figure 9) and for a pillar width of 10 m and a bord width of 14 m.

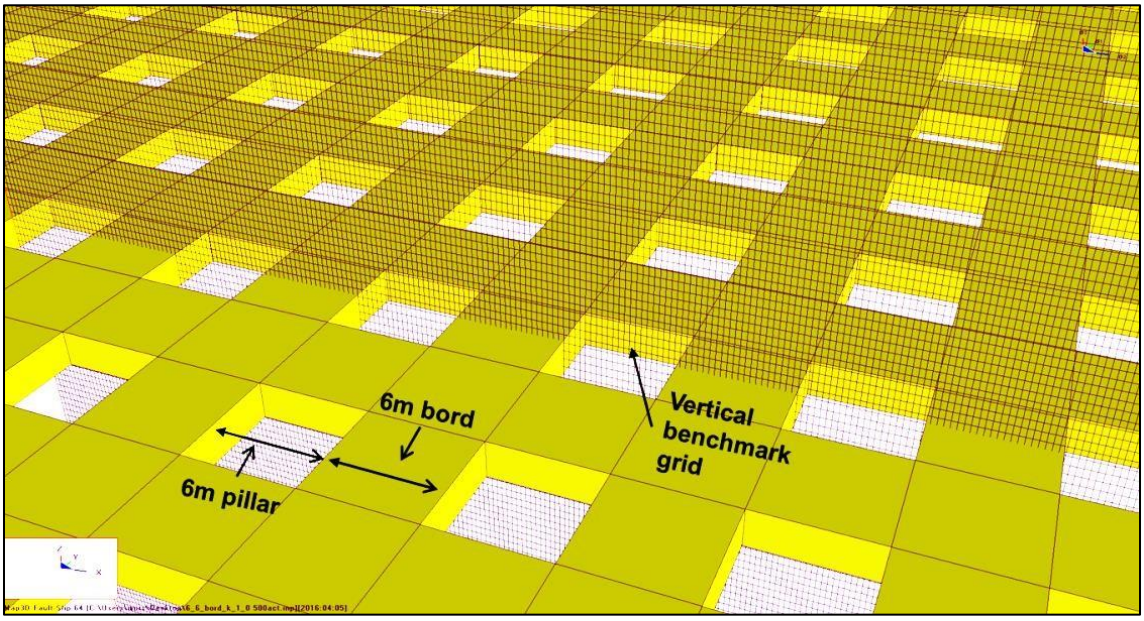

Figure 9: 6 m pillar and 6 m bord model in Map3D

These models were run at depths of 500 m and 1000 m with k-ratios of 1 and 2 for each of the options. The criterion used for the height of failure that would need to be catered for, as shown in Equation 6, is where the fracture of brittle rock will initiate when:

$$\varepsilon_3 \geq \varepsilon_c \qquad \text{Equation 6}$$

Where ε_c is the critical value of extension strain and ε_3 is the minimum extension strain (Stacey, 1981).

In this instance, the critical value was taken as 0.16 millistrains, which is the lowest value obtained in the Brazilian indirect tensile tests. The fractures will form in planes normal to the direction of minimum extension strain (ε_3), which corresponds to the direction of minimum principal stress (Stacey, 1981).

The results are shown in Figure 10 to Figure 17.

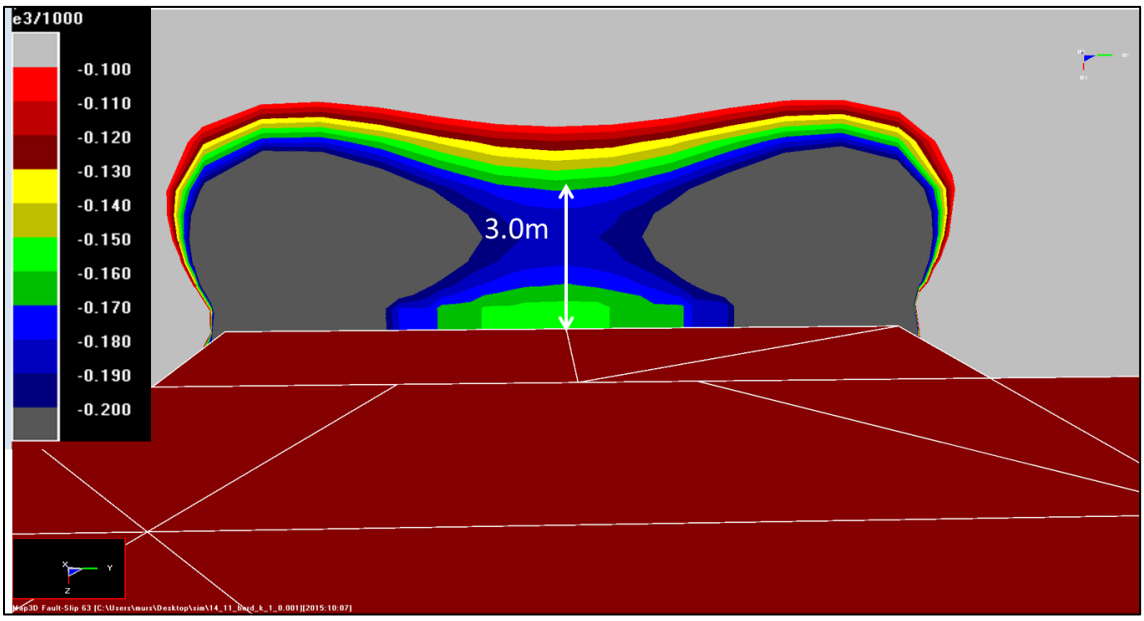

Figure 10: 14 m bord 10 m pillar k = 1 depth 1 000 m

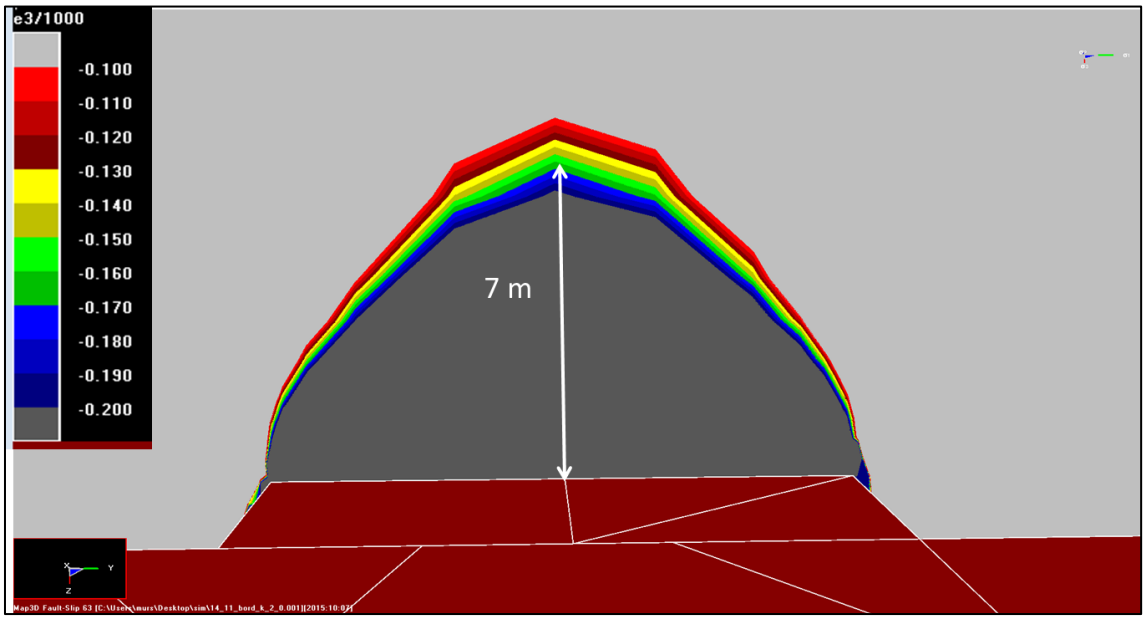

Figure 11: 14 m bord 10 m pillar k = 2 depth 1 000 m

When assessing the 14 m span at 1 000 m the potential failure is related to stress failure that may have occurred at a mine where the fallout thickness was 2 m to 3 m. With k = 2, the fallout height is approximately 7 m. This fallout height does not appear to have occurred at any of the mines visited and therefore it appears that a k-ratio of 2 does not exist at the platinum or chrome mines visited with depths of 1 000 m.

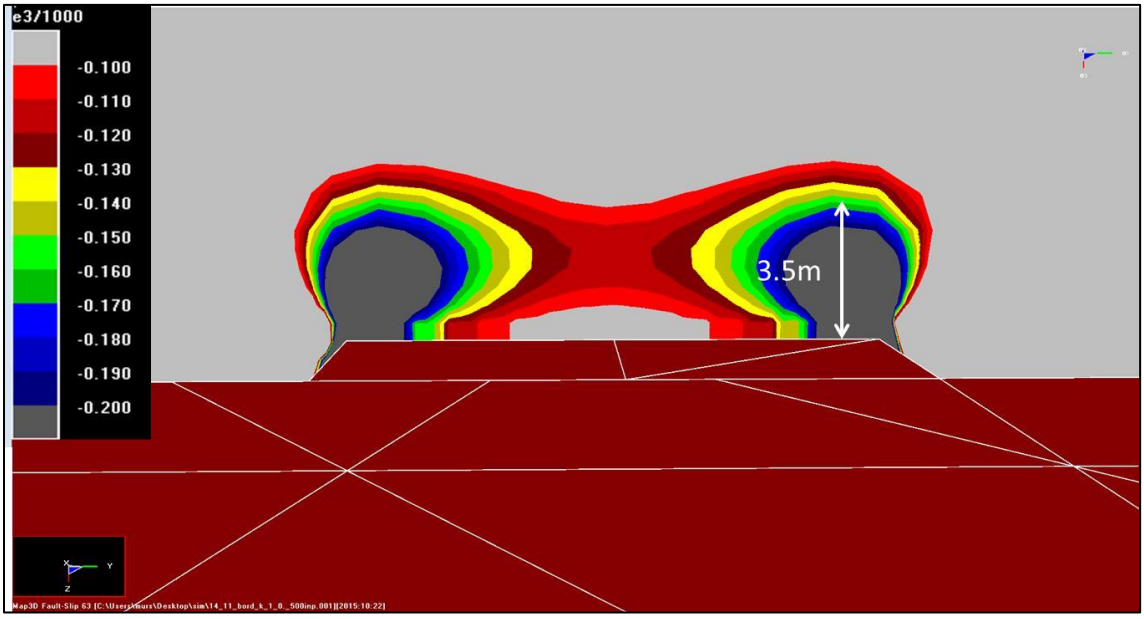

Figure 12: 14 m bord 10 m pillar k = 1 depth 500 m

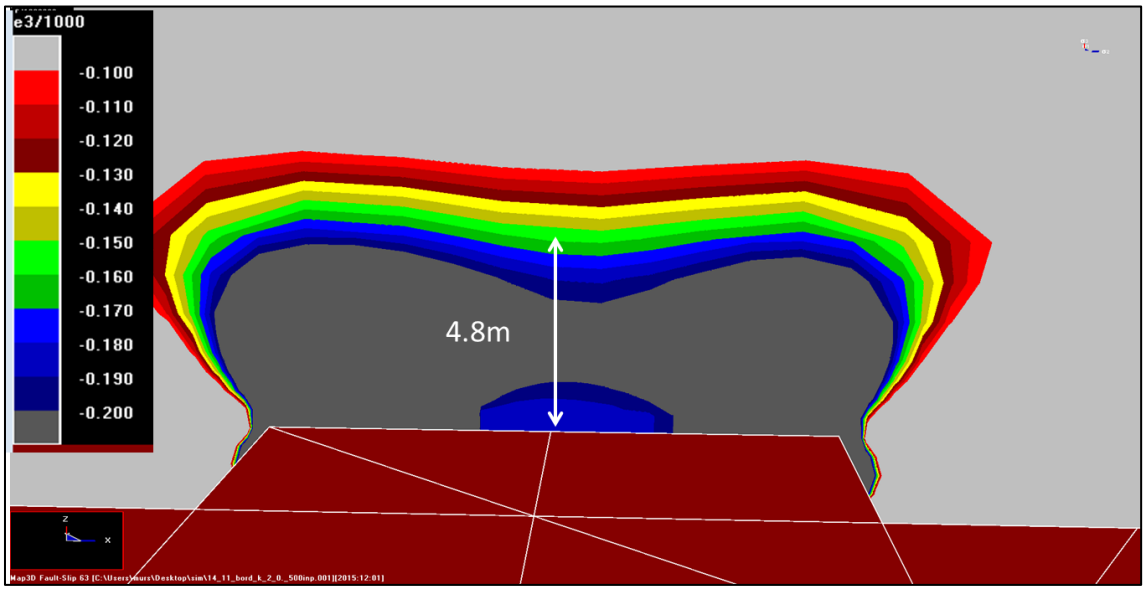

Figure 13: 14 m bord 10 m pillar k = 2 depth 500 m

Figure 13 shows potential stress failure to a height of 3.5 m on the edges of the bord only and it appears that a major collapse could be prevented with the usual support (1.3 m to 1.5 m) that would have contained the centre portion of the bord. However, for a k of 2 it appears that the fallout height is almost 5 m. This type of stress failure has not been identified on any of the mines mining these spans and it may be that a k-ratio of 2 in these areas is an over estimation.

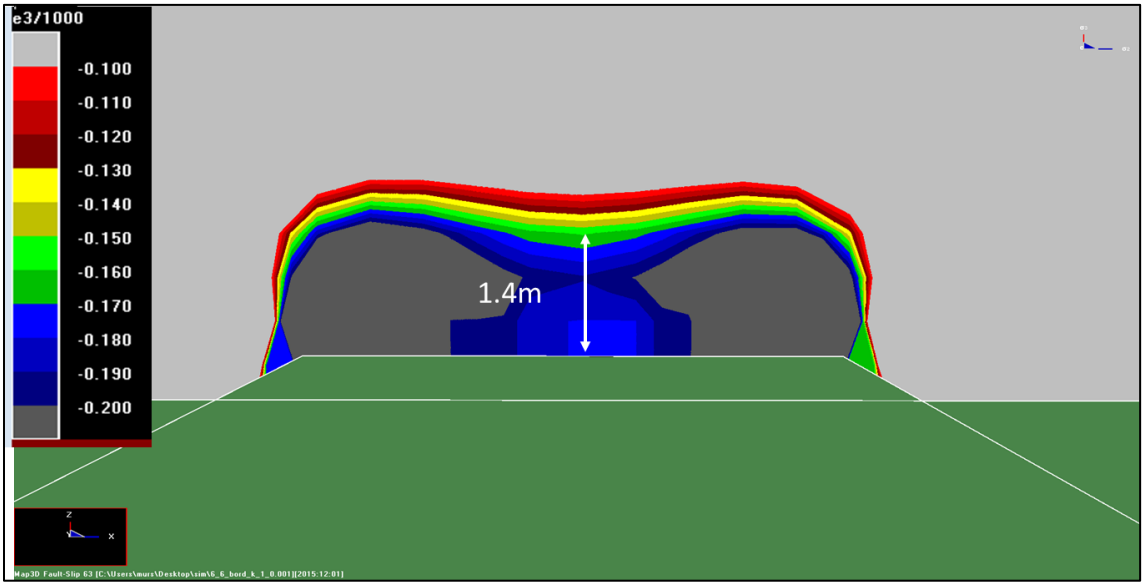

Figure 14: 6 m bord 6 m pillar k = 1 depth 1 000 m

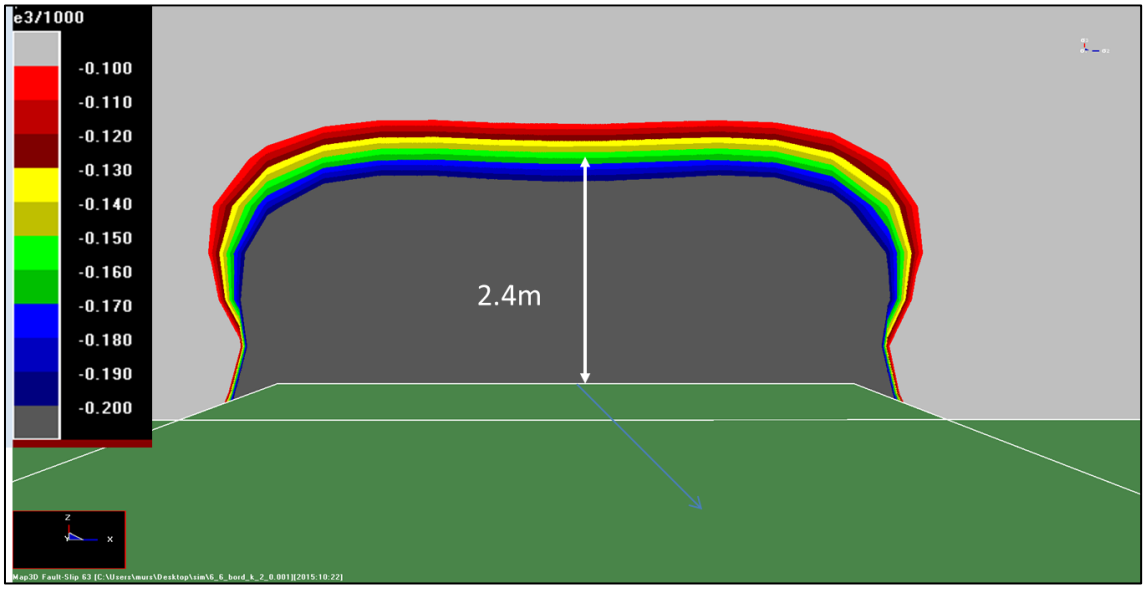

Figure 15: 6 m bord 6 m pillar k = 2 depth 1 000 m

The 6 m bord being mined at 1 000 m depth does not show major failure associated with stress and it appears that this type of failure is being contained by the support system and a k-ratio of 2 is an over estimation of the stress field.

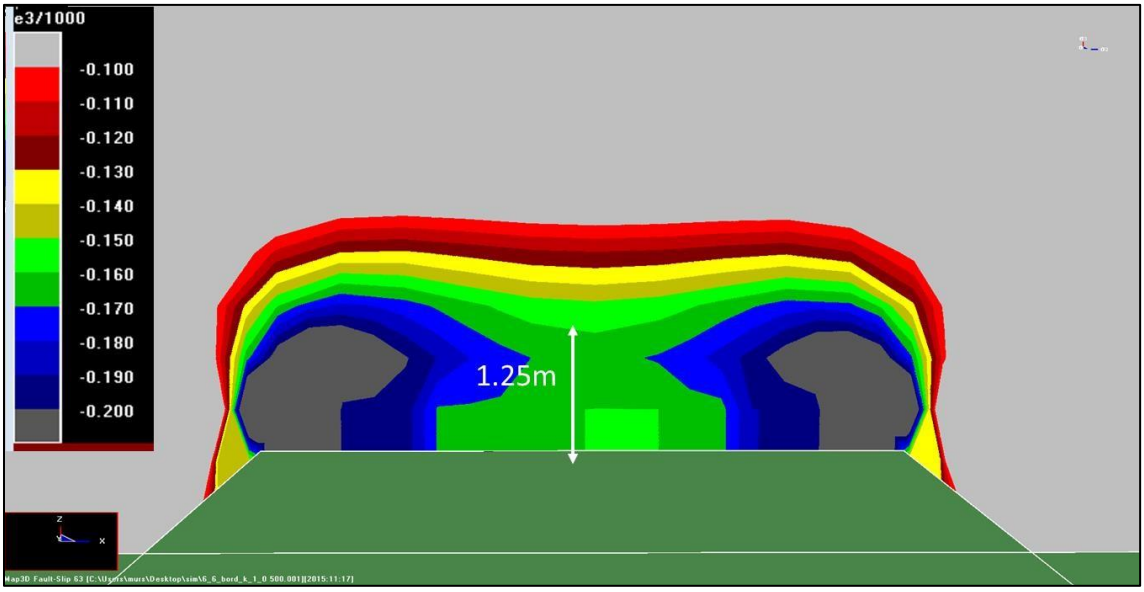

Figure 16: 6 m bord 6m pillar k = 1 depth 500 m

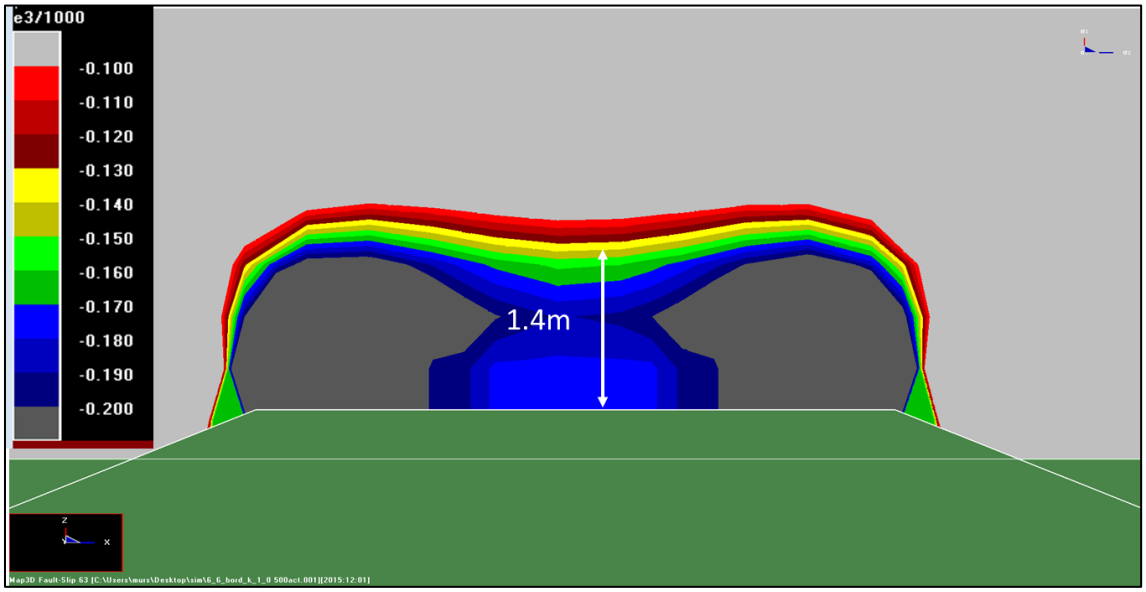

Figure 17: 6 m bord 6m pillar k = 2 depth 500 m

The height of failure shown in Figure 16 and Figure 17 is contained by the support installed at mines operated at this depth and no evidence of stress failure was identified.

Conclusion

The following conclusions can be drawn from the modelling exercise:

1. The height of potential stress failure agrees with observations and data obtained during the site visits.
2. This method appears to be useful in the determination of the height of stress related failure that may occur especially in a brittle rock mass regime where high horizontal stress exists.
3. A k-ratio of 2 at depth appears to be an over estimation of the field stresses when relating the failure that has been observed to the modelled results. It would therefore be expedient to carry out stress measurements where bord and pillar mining is to be carried out.

3.7. Is the bord width stable? (6)

The methodology described thus far will have yielded a result that now needs to be assessed and checked against the original functional requirements, specifications assumptions and constraints. A comprehensive understanding of all relevant interrelating issues is now needed to ensure proper evaluation of the result. If the assessment reveals potential instabilities or indicates more promising alternatives, loop back to the modes of failure / stability analysis stage.

3.8. Implementation (7)

Conclusions and recommendations are results of the implementation process. They will provide a concise statement of the required solution, identify the limitations and indicate how the solution can be successfully implemented. The conclusions and recommendations then need to be communicated effectively to transmit the technical knowledge and put the plan into action.

3.9. Monitoring (8 and 9)

The objective of the monitoring process is twofold. The first objective is to ensure that the design is performing as expected and there are no significant variations from the input parameters used for the design. The second and probably most important objective of monitoring is to identify low-angle features, potential weak layers and any form of deformation that may be occurring. Monitoring is usually done by rock engineering personnel using the following:

- Camera with boreholes.
- Digital radar in borehole (experimental stage).
- Ground Penetrating Radar (useful where chrome stringers need to be identified).
- Mapping of rock mass conditions where necessary.
- Additional lighting to assist in the identification of weak areas.

The crux of the matter is to ensure that sufficient geotechnical information is obtained and available at all times.

During the research completed for this project for this project, it emerged the systematic use of borehole cameras appears to be the most efficient methodology utilised to identify problematic joints, including low angled (Esterhuizen, 2014 and mine-specific information).

Finally if potential design failure is identified, the methodology must be revisited and redesigned.

Chapter 4 Conclusions and recommendations

The proposed design methodology should be used during all stages of the mining process, from prefeasibility to final design and implementation, and when compiling codes of practice to combat rockfall accidents.

The development of a proper engineering approach for the design of maximum stable bord widths should be holistic and consider all types of failure mechanisms that may occur, i.e. intact rock mass, structural, beam and rock mass.

Although most FoG incidents / accidents are associated with failure along geological structures, most mines do apply a design methodology based on structural analysis for bord and pillar operations.

Bord stability is dependent on the identification of low angle features, potential weak layers and deformation. Additional support is required to contain potential failure associated with these rock mass instabilities.

In a few cases, complicated numerical analysis programs are used on an ad hoc basis to assess structurally controlled panel stability.

Bord width must be tailored to the specific rock and engineering environment, taking into account joint orientation, strain (depth) and notably the potential hazards associated with low-angled (≤ 60°) joints. A succinct description of the solution must be provided including any limitations or restrictions associated with the proposed method.

There is no definitive rule to guide the maximum width of a bord. Experience has shown that even in a 3 m to 6 m span, a major rockfall with a fallout height in excess of the width of the excavation can and does occur.

It should be noted from the structural analysis that reducing the span does not eliminate the occurrence of large or small rockfalls and additional measures will therefore be required to manage the risk. The frequency of large rockfalls can be further reduced by introducing longer and stronger bolts, such as 38 tonne cable anchors. However, since these large rockfalls are infrequent, it will invariably be more cost effective to implement an effective monitoring programme (e.g. borehole camera or ground penetrating radar) and only install the additional support where it is required.

Beam failure is an important assessment but the typical bord and pillar layouts will “pass” such a test. It is only in locally poor ground conditions that the span may fail the Voussoir arch stability test.

Elastic numerical modelling has proven to be useful to determine the height of stress related failure that may occur especially in a brittle rock mass regime where high horizontal stress exists.

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