



Application of an empirical method and numerical modelling to the Merensky Reef crush pillar stability

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Dates:

Received: 23 Aug. 2023
Revised: 14 Nov. 2024
Accepted: 13 Dec. 2024
Published: February 2025

How to cite:

Le Bron, K.B., van Aard, T. 2025 .
Application of an empirical method and
numerical modelling to the Merensky Reef
crush pillar stability. *Journal of the Southern
African Institute of Mining and Metallurgy*,
vol. 125, no. 2 pp. 87–98

DOI ID:

<https://doi.org/10.17159/2411-9717/3060/2025>

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Abstract

The empirical pillar strength formulae have been widely, and in most cases successfully, applied to design underground pillars for mines in the Bushveld Igneous Complex (BIC). This paper presents the case for combining the empirical method with numerical modelling when it comes to crush pillar stability analyses. The benefits include an understanding of potential failure mechanisms, which may then be accounted for in the designs. In addition, the impact of weak layers, such as shear zones, the impact of the dip of the orebody, differences in layouts, such as holing widths and the presence of advanced strike gulleys (ASGs) immediately adjacent to the crush pillars, and stress regime (k-ratio) may be studied using numerical modelling, as it is not accounted for in the empirical method.

Keywords

cube strength, numerical modelling, stress regime, empirical method, crush pillar stability

Introduction

The empirical pillar strength formulae have been widely, and in most cases successfully, applied to design underground pillars for mines in the Bushveld Igneous Complex (BIC). These formulae have limitations in the sense that it does not account for:

- (Site specific rock mass conditions in the formulae, i.e., weak layers such as the presence of shear zones at the top of the UG2 chromitite seam has such a persistent weak clay layer above or at the top contact, and bottom (the MG1 on the western limb of the BIC has such a persistent shear/fault at its bottom contact) of tabular orebodies,
- A stress regime in the formulae (the dip of the orebody impacts the normal and shear loading of the pillars),
- The complexity of layout in the formulae, i.e., the difference in heights on either side of the pillars, e.g., the presence of an advanced strike gully (ASG) next to the pillar,
- Any method for determining the cube strength. It was recommended to be a factor of the uniaxial compressive strength (UCS) of the pillar material; however a combination of different rock types may be present within the pillars, etc. Numerical models however allow for an investigation of site-specific rock mass conditions, complex layouts, different stress regimes, etc., which is useful in understanding the impact of these parameters on the pillar behaviour. The aim of the empirical pillar strength formulae is to ensure the design of stable pillars, which would not fail prematurely. However, there have been reports of crush pillar bursts on the Merensky Reef horizon, which is on the opposite end of the scale resulting in the following conundrum:
 - if the pillars are too small, they may fail prematurely, and
 - if they are too large, they may result in rock bursts.

This study was commissioned to study specific underground panel and pillar layouts, which have been based on the empirical pillar strength formulae, to determine the following:

1. The cube strength of Merensky Reef.
2. The impact of pillar height on crush pillar stability.
3. The impact of ASG versus siding on crush pillar stability.
4. The impact of pillar holing width on crush pillar stability.

Following here is a discussion about the learnings of this study.

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Literature review

Pillar strength can be estimated from empirical equations that have been developed by observing both failed and stable pillar configurations. The design of stable pillars requires that both the strength and loading of the pillars be known. In addition, an appropriate safety factor should be selected to ensure that the variability and uncertainty of the pillar strength and loading is accounted for. In the case of regular arrays of flat lying pillars, the load can be estimated by the tributary area method (Salamon and Munro, 1967), or if the layout is more complex, estimates of average pillar loading can be obtained from numerical models (Brady and Brown, 1985). Following the Coalbrook North Colliery disaster in 1960, where the entire mine collapsed as a result of a cascading pillar failure, a study was commissioned to determine what the causes were of pillar failures, as well as to compile guidelines for future pillar designs. An empirical approach was followed, which involved studying data from a total of 125 pillars in South African coalfields, of which 27 constituted collapsed pillars. Based on a statistical appraisal of the data, Salamon and Munro (1967) developed a pillar strength formula, which is presented below:

$$\sigma_s = K w^\alpha / h^\beta \quad [1]$$

where:

σ_s = strength of coal pillar
 K = statistically determined strength of a unit cube of coal
 w = pillar width
 α, β = empirical constants
 h = height of a pillar

A factor of safety (FOS) of 1.6 was recommended by Salamon and Munro (1967) (ratio of pillar strength to pillar stress). Several empirically based pillar strength equations have since been developed for hard rock mines (Hedley and Grant, 1972; Von Kimmelman et al. (1984); Lunder and Pakalnis, 1997). Hedley and Grant (1972) specifically applied the approach adopted by Salamon and Munro (1967) to hard rock mines. They monitored 28 pillars at uranium mines in Canada, of which only three were defined to have failed. Based on this, Hedley and Grant (1972) developed a pillar strength formula for hard rock mines, which is presented below:

$$\sigma_s = K w^{0.5} / h^{0.75} \quad [2]$$

where:

σ_s = strength of hard rock pillar
 K = statistically determined strength of a unit cube of rock
 w = pillar width
 α, β = empirical constants
 h = height of a pillar

In the case of crush pillars, a factor of safety (FOS) of less than 1 is recommended as the pillars are designed to crush in a stable manner. Width-to-height ratios of crush pillars employed in the South African gold and platinum mines generally vary between ~1.3 to ~2.5. The schematic shown in Figure 1, highlights guidelines for pillar designs for mines located in the Bushveld Igneous Complex (Anon, 2006).

Esterhuizen (2006) studied the impact of width-to-height ratio on pillar stability, where he showed that for the presented case histories, the pillar strength becomes highly variable as the width-to-height ratio decreases (two pillars failed at width-to-height ratios of 2 and above). Esterhuizen (2006) defined pillars with width-to-height ratios of less than 1.0 as slender pillars, which is less than the

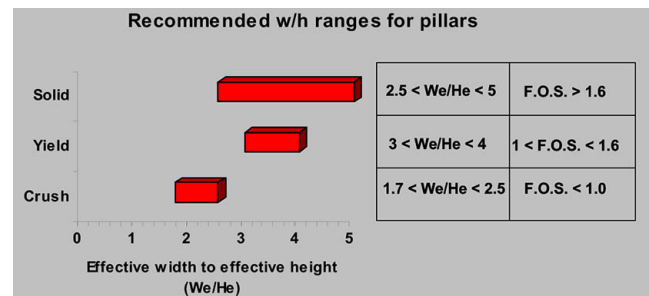


Figure 1—Guidelines for pillar designs for mines located in the Bushveld Igneous Complex (Anon, 2006)

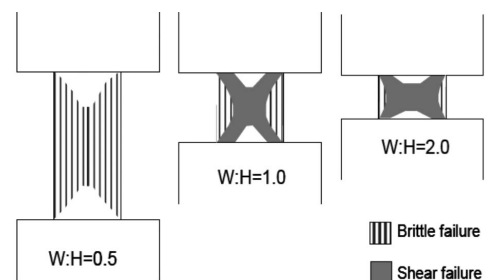


Figure 2—Sections through the centre of pillars with different width-to-height ratios showing the extent of brittle and shear failure of the rock mass predicted by numerical modelling

minimum recommended width-to-height ratio for crush pillars. The extent of brittle and shear failure in pillars with width-to-height ratios of 0.5, 1.0, and 2.0 are presented in Figure 2 (Esterhuizen, 2006), illustrating the increasing role of brittle failure as the width-to-height ratio decreases. Failure of the wider model pillars is initiated by brittle failure around the outside of the pillar, which commences when the stress in the outer skin of the pillar exceeds the brittle rock strength. The brittle failure process continues as the pillar load increases. Note that the progression of potential failure within the crush pillars is expected to follow that of the schematic for a width-to-height ratio of ~2 (Figure 2).

One of the main findings of this study was that the standard deviation of the strengths of slender pillars was 25.4%, while it was 7.8% for the wider pillars. The variability can be caused by several factors, which can include uncertainty of the actual rock strength, uncertainty of the pillar stress, variations in the degree and severity of jointing, a variation in the bedding characteristics, and the presence of weak layers in the pillars.

Malan (2010) surmised that two different methods to determine the strength of pillars are currently used namely, empirical equations derived from the back analyses of both failed and stable cases and numerical modelling tools with appropriate failure criteria. Both techniques have their limitations and additional work is required to obtain a better understanding of pillar strength. He concluded that neither empirical techniques nor numerical modelling currently provide a solid basis to conduct pillar design. He further recommended that both these techniques be utilised when confronted with pillar designs to obtain the best possible insight into the problem.

Stability criteria

The vertical axial strain at failure may be used to predict potential failure in the rock mass where similar unconfined conditions may exist. The edges and corners of the underground pillars may experience similar unconfined compression, which means that

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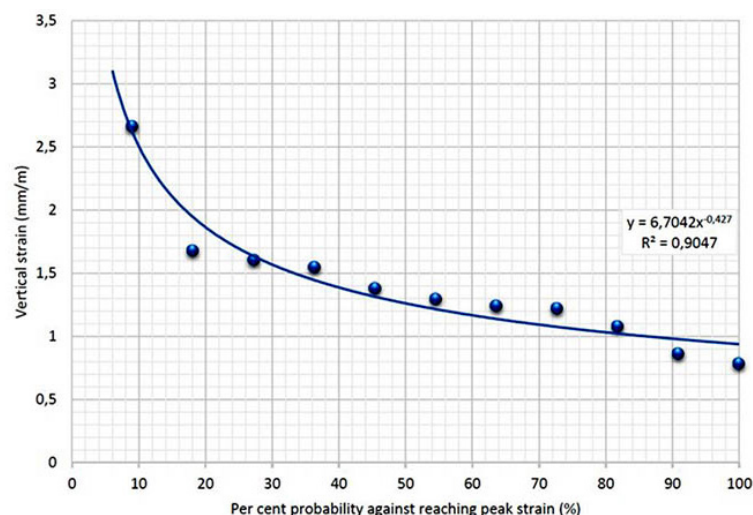


Figure 3—Graph showing probability against reaching failure at peak stress as a function of axial strain based on UCS tests

Pillar	Side	Height (m)	Width (m)	Length (m)
B	1	0,93	3,35	5,53
	2	1,52	3,86	6,56
	3	0,84	4,09	6,3
	4	1,13		
AVERAGE		1,24	3,77	6,13
C	1	0,63	3,56	5,76
	2	1,9	4,26	6,32
	3	0,89	4,06	6,21
	4	1,05		
AVERAGE		1,38	3,96	6,10
D	1	0,81	2,84	5,95
	2	2,24	3,55	6,05
	3	1,13	4,01	6,22
	4	0,97		
AVERAGE		1,61	3,47	6,07
F	1	0,92	3,94	6,24
	2	1,53	4,2	6,39
	3	1,08	4,49	6,34
	4	1,07		
AVERAGE		1,28	4,21	6,32
G	1	0,67	5,22	4,13
	2	2,50	5,67	4,71
	3	1,11	5,41	5,9
	4	1,31		
AVERAGE		1,76	5,43	4,91

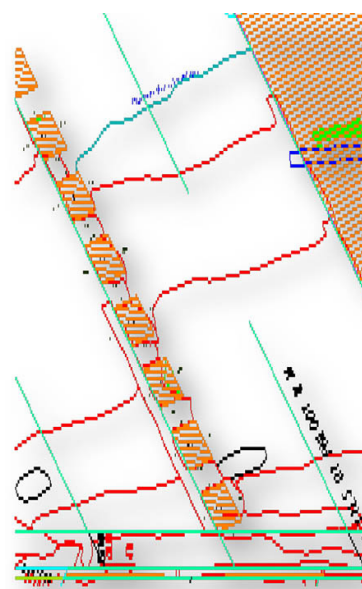


Figure 4—Pillar heights and location

the vertical axial strain may be applied as a criterion in numerical simulations, to determine the onset of pillar fracturing, as a precursor to slabbing and spalling of the pillar sides. The best fit power law relationship is shown in Figure 3 for the 11 site-specific test samples tested, which has a coefficient of determination (R^2) of ~0.91. Based on the analysis, the minimum axial strain value of ~0.9 mm/m may be used as a criterion to indicate in which zones the peak strength had been exceeded, i.e., which zones are in the post-peak strain-softening phase.

Elastic material properties were initially assigned to the rock mass simulated in the numerical models, in order to determine the minimum deformation levels predicted within the crush pillars. It is assumed that if the deformation levels predicted for the elastic material properties result in vertical strain exceeding the strain criterion of 0.9 mm/m (from Figure 3), the pillars would be considered crush pillars and would not pose a risk of bursting as the actual resultant in situ inelastic deformation would be greater than the elastic deformation.

In the non-linear numerical models, it was assumed that if the model does not converge (reach equilibrium), the crush pillars will

continue to deform and eventually disintegrate. The crush pillar dimensions that resulted in the models not converging were then defined as not suitable for the mining conditions. Furthermore, this method was also applied to quantify strain values for disintegration of the crush pillars. Unfortunately, there is no technical publication that could be cited as a benchmark against which this novel approach can be tested.

Merensky Reef cube strength estimation

A back analysis of the insitu Merensky reef cube strength was conducted, using the Hedley and Grant (1972) formula, which has been used to design most of the Merensky Reef pillars of the Bushveld Igneous Complex. Detailed information on pillar dimensions and rock wall conditions was available after a comprehensive data gathering exercise in preparation for this back analysis had been conducted. The stoping layout incorporating the stope panels and crush pillars was modelled elastically to a fine discretised mesh of 0.5 m (vertical) × 0.5 m (strike) × 1 m (along dip). Figure 4 presents the crush pillar numbering convention and actual pillar heights measured at all four exposed sides (for ease of

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reference, a plan view of the stope panel and annotated crush pillars are presented in Figure 1, together with the pillar dimensions). Pillar A had not yet been formed at the time of the study and was therefore not incorporated into the study. Pillar B was also not completely formed; however all four sides had been exposed and was therefore included in the study. Pillar E was significantly undersized and was also excluded from the back analysis. The average pillar heights were calculated by averaging the three sides of the pillars referred to as 1 (holing), 3 (holing) and 4 (panel). The value obtained from this calculation was then averaged with pillar side 2 (ASG side). This method assumes a pillar height, which is unlikely to overestimate the effective pillar height. The pillar widths and pillar lengths were taken at 3 locations across the pillars, based on the survey measurement data. The pillar widths were averaged based on three measurements across the pillar, which include the measurements at the pillar ends and one in the middle of the pillar.

Underground data collection and modelling results

Figure 5 to Figure 11 present the modelled stress in the crush pillars, as well as photos showing the condition of the pillar walls. Photos are shown for the pillar closest to the face (Pillar B) and the pillar furthest from the face (Pillar G), for comparison. The results generally show that the pillar stress is highest where the pillar height is smallest, i.e., on the ASG side the pillar stresses are generally lower due to the increased height of the ASG. The photos indicate that most of the pillars are extensively fractured and may have crushed. Figure 11, which details a graph of the average modelled pillar stress (APS) for the crush pillars, shows that the stress on the pillars closer to the panel face is generally lower than the pillars in the back area. It must be noted that the stress on pillars D, F, and G tends towards an APS of approximately 120 MPa, whereas pillar C has an APS of ~110 MPa and pillar B, which has not yet been fully created, has an APS of ~80 MPa. As mentioned previously, pillar E

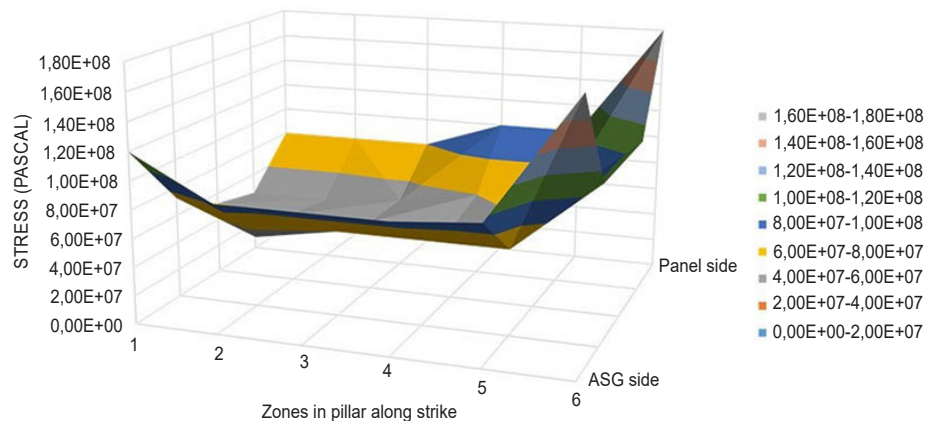


Figure 5—Schematic showing the stress in crush pillar B



Figure 6—Photo of pillar B, side 1



Figure 7—Photo of pillar B showing intense fracturing

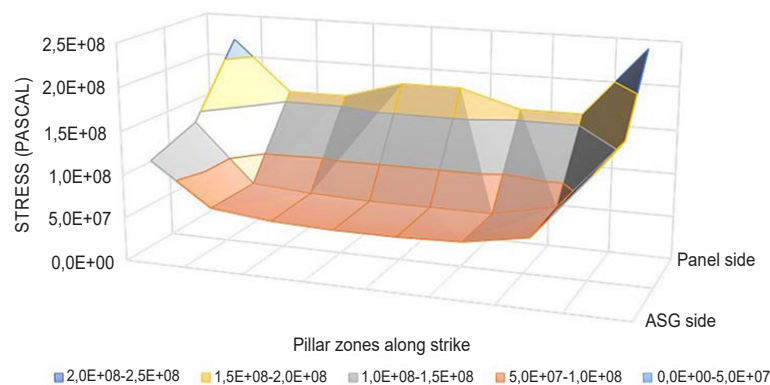


Figure 8—Schematic showing the stress in crush pillar G

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Figure 9—Photo of pillar G, side 1



Figure 10—Photo of pillar G showing scaling

was significantly undersized and resulted in a significantly higher APS value of ~155 MPa (stress is calculated by dividing the load with the area, i.e., smaller area of Pillar E resulting in higher stress). The back analysis is based on the reasonable assumption that the crush pillars are either at an FOS of 1 or very close to it (the sides of the crush pillars have already fractured, whilst Pillar B has not been fully created on the face). It is therefore reasonable to assume that these pillars would have an FOS of ~1.s.

The strain and stress profiles across pillar B are shown in Figure 12. Figure 13 shows the locations in pillar B, where the stress and strain values were taken. The strain criterion of 0.9 mm/m (derived from Figure 3) is used to locate the critical stress in Figure 13. A critical stress of approximately 47 MPa is suggested for pillar B, based on the FLAC3D results.

The same approach has been applied to pillar C to pillar G. Figure 14 presents the relationship between stress and strain for pillar C to pillar G. Pillar E has not been included due to its irregular shape. It should be noted that the lowest strain values in pillar C to pillar G exceeded the strain criterion of 0.9 mm/m, however the relationship between stress and strain for each pillar was extended to the intersect the strain criterion of 0.9 mm/m, from which the critical stress was inferred. The critical stress level in pillar C is ~56 MPa. The critical stress level in pillar D is ~44 MPa. The critical stress level in pillar F is ~37 MPa. The critical stress level in pillar G is ~43 MPa.

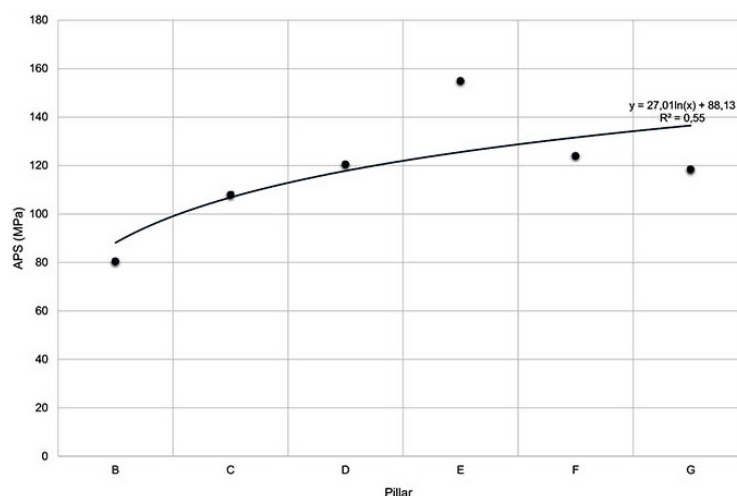


Figure 11—Schematic showing the average modelled APS in the crush pillar

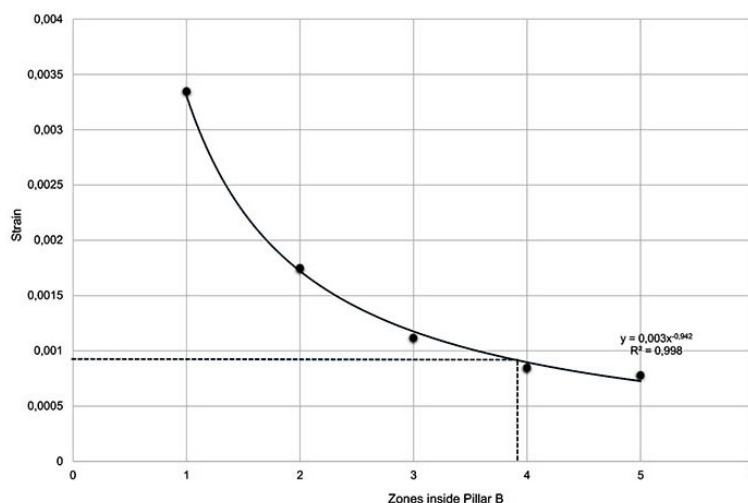


Figure 12—Strain profile across pillar B showing critical strain level

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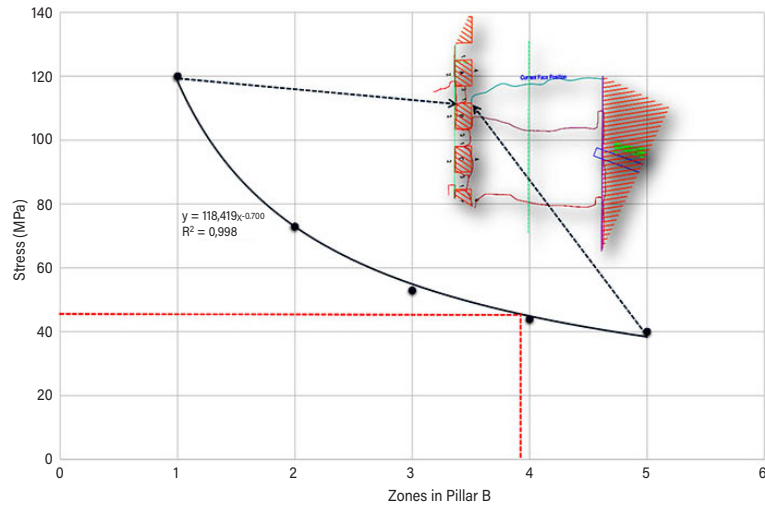


Figure 13—Stress profile across pillar B showing the critical stress level (note that the pillar has not holed)

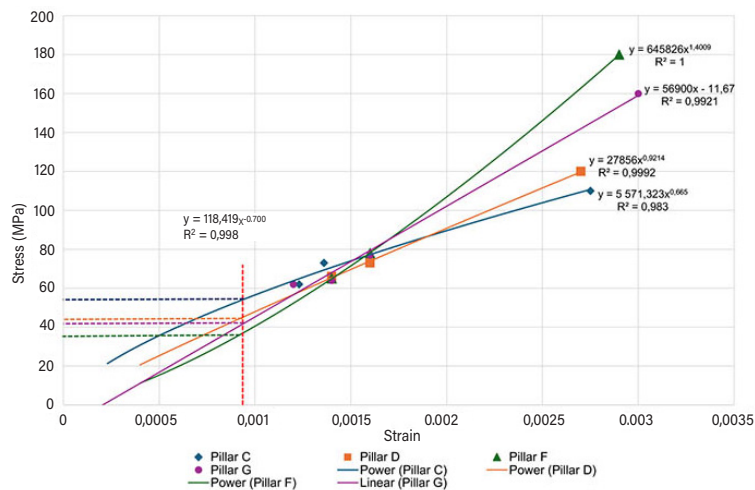


Figure 14—Stress-strain relationship in pillar C, pillar D, pillar F, and pillar G

Table I					
Summary of back analysis of insitu cube strength (K-value)					
	Pillar B	Pillar C	Pillar D	Pillar F	Pillar G
Critical Stress at 0.9 mm/m vertical strain	47	56	44	37	42
Back Analysed K-Value (MPa)	78	88	58	65	64

Back analysis of the Merensky Reef cube strength (K-Value)

For this study, the Hedley Grant (1972) formula (Equation 2) was applied to back analyse the insitu cube strength (K) for the Merensky Reef. As explained previously, once the critical stress was determined for the critical vertical strain of 0.9 mm/m, the critical stress was then determined at those locations. When the FOS is equal to 1, the pillar stress would be equal to the pillar strength. These critical stress values were then used in the Hedley Grant formula by substituting the pillar strength (PS) to determine the cube strength (K-value). The heights were calculated by averaging three sides of the pillar including the holings and the down-dip panel. The average of the height obtained and the height in the ASG were used to calculate the pillar height. The calculated K-values based on the back analysis of each pillar are presented in Table 1

Summary of the Merensky cube strength back analysis results

The FOS for the pillars based on the APS and H-G formula using: (i) a minimum of 58 and (ii) the individual back-calculated K-values, is shown in Table 2 and Table 3 respectively.

The skin of the pillars for all cases are predicted to fracture by the modelling (also shown in the pictures), which is why the inferred critical stress at failure was used. The results indicate that the face area has a major effect in reducing the stress level on the pillars, which is why the FOS is greater than 1 for pillar B. The minimum K-value of 58 MPa seems realistic based on the FOS calculations, and the underground visual observations (especially for pillar B, which has not been fully created).

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The impact of mining height on crush pillar behaviour

The following combinations of crush pillar dimensions were simulated in FLAC3D (note that no ASG was modelled next to the crush pillars):

- 6 m (length) × 2 m (width) × 1.0/1.5 m (height)
- 6 m (length) × 2.5 m (width) × 1.0/1.5 m (height)
- 6 m (length) × 3 m (width) × 1.0/1.5 m (height)
- 6 m (length) × 4 m (width) × 1.0/1.5 m (height)
- 6 m (length) × 5 m (width) × 1.0/1.5 m (height)

The decline protection pillars were included in these models to realistically determine resultant deformation and strain of crush pillar instability. Plots of the vertical strain block contours predicted within the simulated crush pillars are shown in Figure 15, also presented in graph format in Figure 16. The results show that the strain that is predicted to occur in the crush pillars, will

exceed the criterion of 0.9 mm/m, which is required for fracturing to occur. Nowhere within the simulated crush pillars will the strain at failure not be reached, which for all intents and purposes implied that pillar bursting, was unlikely to occur (provided that the actual underground mining layout comply with the design).

In order to reduce run-time, separate models were set up to carry out analyses on crush pillar widths. In these models, only two of the three regional pillars were simulated with a single belt drive, as the focus was on the crush pillars rather than the service excavations. It is therefore not directly comparable to the previous set of results. The strain criterion was used in this sensitivity analysis. The results show that the 2 m and 4.5 m wide crush pillars are predicted to crush at a pillar height of 1.5 m (see Figure 17 and Figure 18). However, if the regional pillars are excluded from the model, the 3 m wide crush pillar deforms significantly more. The models simulating the crush pillars at widths of less than 3 m

Table 2

FOS based on in situ cube strength (K-value) of 58 MPa

	APS	Strength	FOS
B	81	96	1.18
C	109	91	0.83
D	121	76	0.62
F	124	99	0.80
G	118	84	0.71

Table 3

FOS based on individual in situ cube strength (K-value) of each pillar as back calculated

	APS	Strength	FOS
B	81	129	1.59
C	109	138	1.26
D	121	76	0.62
F	124	111	0.89
G	118	93	0.79

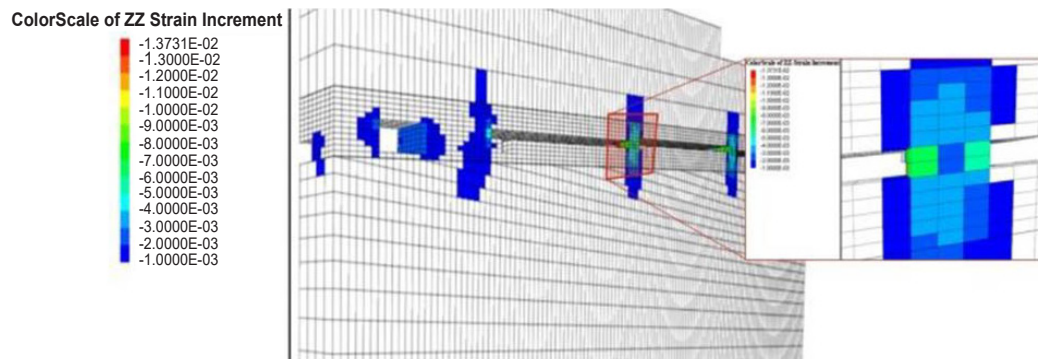


Figure 15—Schematic showing the vertical strain contours predicted to occur in the crush pillars (1 m height)

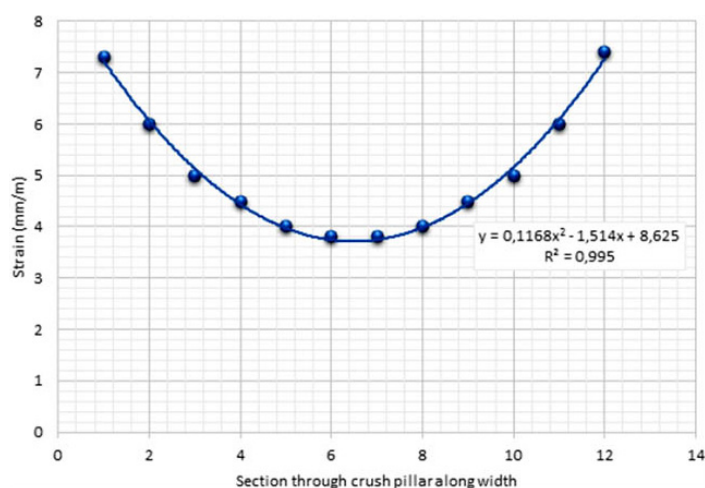


Figure 16 —Graph showing the vertical strain contours predicted to occur in the crush pillars (1 m height)

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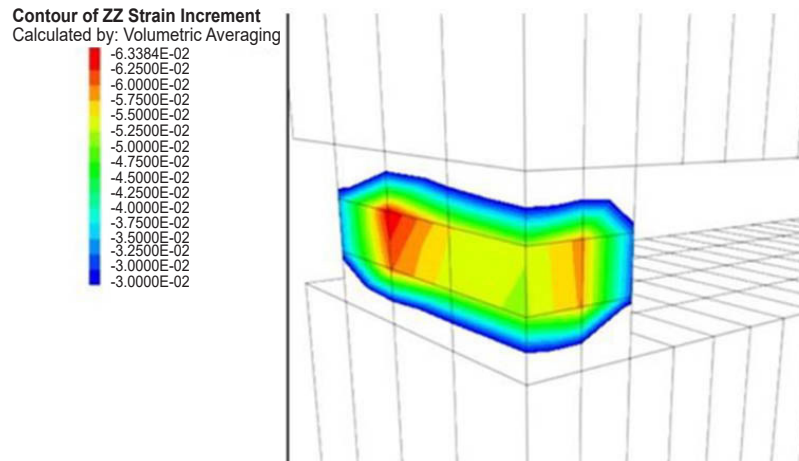


Figure 17—Schematic showing the vertical strain contours predicted to occur in the crush pillars (6 m x 2 m x 1.5 m (height)) with regional pillar

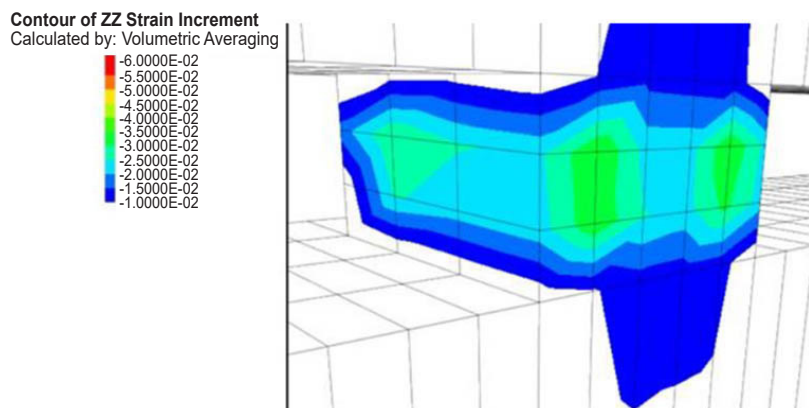


Figure 18—Schematic showing the vertical strain contours predicted to occur in the crush pillars (6 m x 4.5 m x 1.5 m (height)) with regional pillars

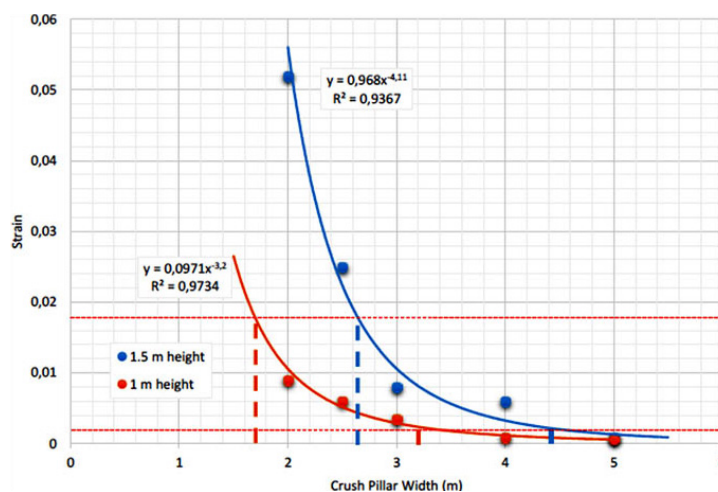


Figure 19—Graph showing the relationship between vertical strain contours and pillar width for 1 m and 1.5 m pillar heights (stope widths), respectively

with no regional pillars included, do not converge, indicating that the crush pillars are predicted to continuously deform, leading to disintegration of the crush pillars.

Based on the results, when the strain in the core of the crush pillars exceeded a value of ~18 mm/m, the models did not converge, which was defined as the criterion indicating crush pillar failure/collapse. The criterion of 0.9 mm/m (indicating brittle failure of the crush pillars), together with the strain criterion of ~18 mm/m

(indicating collapse of the crush pillars) was applied to determine lower and upper limits of crush pillar widths for pillar heights of 1 m and 1.5 m (Figure 19).

The derived upper and lower width dimensions for crush pillars of different mining heights are summarised in Table 4, which are:

- 1.7 m and 2.6 m for a 1 m stopping height, and
- 3.2 m and 4.4 m for a 1.5 m stopping height.

Table 4

Derived upper and lower width dimensions for crush pillars of different mining heights of 1 m and 1.5 m

	Pillar Height / Stope width of 1 m	Pillar Height / Stope width of 1.5 m
Pillar width (minimum)	1.7 m	2.6 m
Pillar width-to-height ratio (minimum)	1.7	1.73
Pillar width (maximum)	3.2 m	4.4 m
Pillar width-to-height ratio (minimum)	3.2	2.93

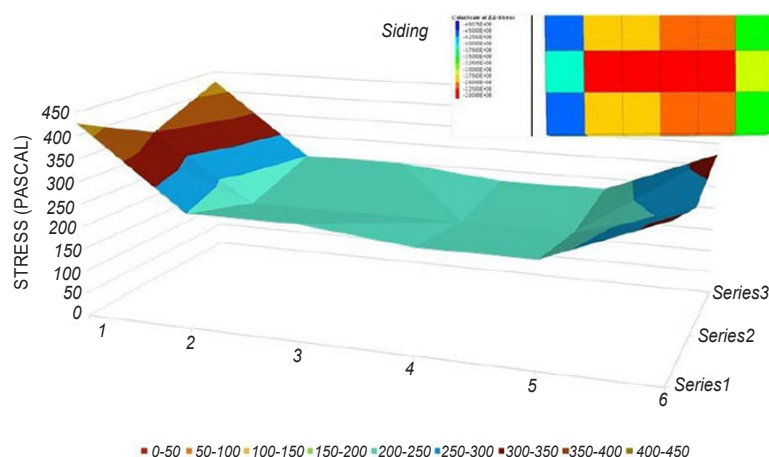


Figure 20—Schematic showing the vertical stresses predicted to occur in the crush pillars where a siding is excavated

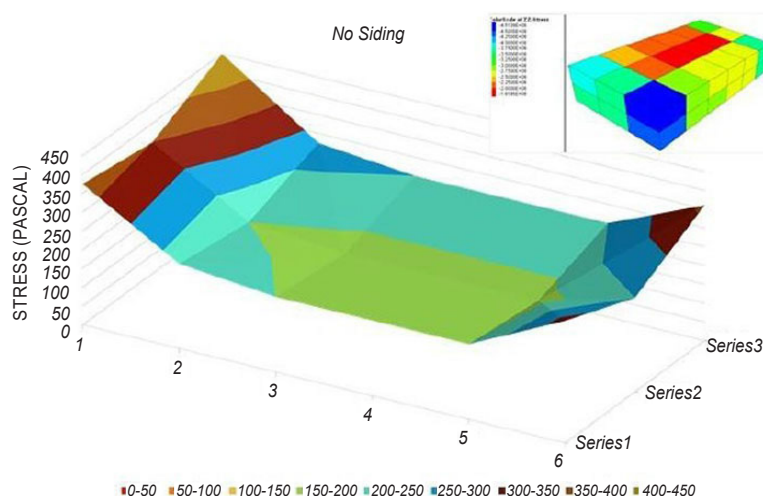


Figure 21 —Schematic showing the vertical stresses predicted to occur in the crush pillars where no siding is present (note the differential stress concentration)

The effect of ASG versus siding on crush pillar behaviour

The impact of the ASG versus the siding on the down-dip side of the stope panel was investigated by simulation within the same model: (i) stope panel with an ASG but without a siding and (ii) stope panel with an ASG, but with a siding. Elastic material properties were assigned to the rock mass for this comparative assessment. An orebody dip of 12° was assumed. The height of the ASG was modelled at 2.4 m, with the stope width being 1.2 m.

The results show that where a siding is cut, the stresses that form within a crush pillar is symmetrical, whereas the stresses in a crush pillar where no siding is present, is not symmetrical (see plots of vertical stresses within crush pillars for the scenarios in Figure 20 and Figure 21).

The presence of an ASG with a modelled height of 2.4 m, results in a significant stress drop on the down-dip side of the crush pillar, resulting in the uneven stress profile along dip. The results of this uneven stress profile are:

- Differential movement in the dip direction (see plots in Figure 22 and Figure 23). The differential movement may lead to the formation of shear fractures, along which dilation may occur, which will ultimately reduce the crush pillar stability.
- The potential for fracture formation, which may lead to sidewall slabbing is highest on the up-dip side for the case where no siding is present. This may lead to pillar scaling, resulting in higher loads being carried by the remaining portion of the crush pillar. However, due to the height of the

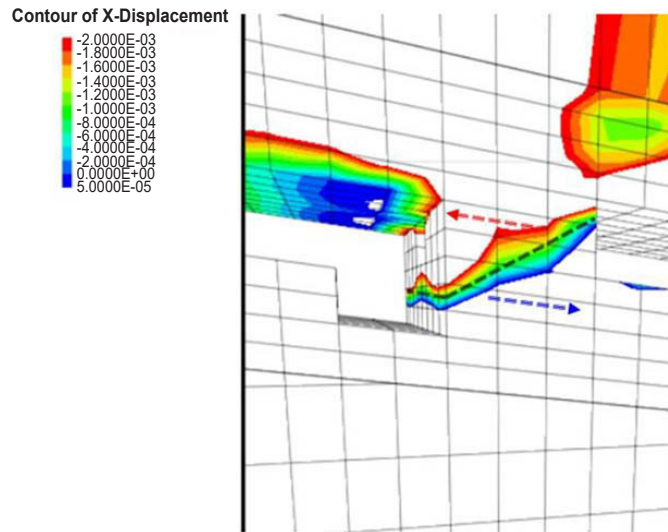


Figure 22—Schematic showing the displacement contours along dip predicted to occur in the crush pillars where no siding is present (note the differential movement may lead to the formation of shear fracture which will reduce the crush pillar stability)

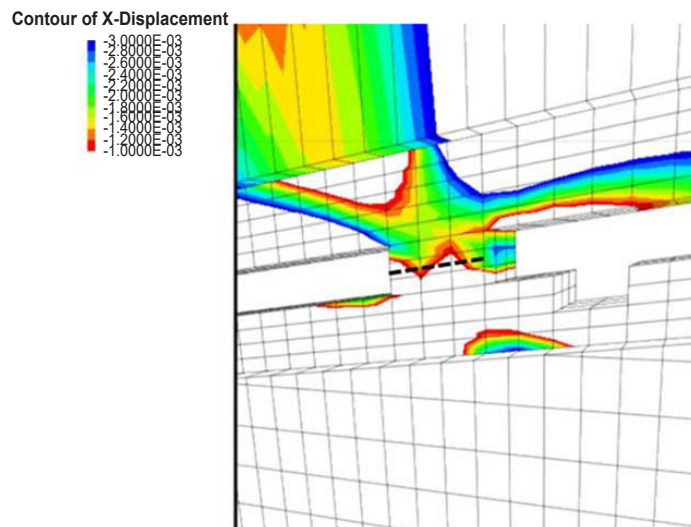


Figure 23—Schematic showing the displacement contours along dip predicted to occur in the crush pillars where a siding is present (no differential movement predicted within the pillar where sidings are present)

strike gully being ~2.4 m, the crush pillars will have a reduced capacity to absorb the additional load that is transferred due to scaling on the up-dip side of the pillar. This scenario may ultimately lead to disintegration of the crush pillars due to excessive slabbing and continuous scaling.

The impact of ventilation holing width on crush pillar behaviour

The impact of the ventilating holing width was investigated through numerical modelling as part of this study. Figure 24 presents the locations of vertical stress profiles through a 4 m (strike) × 3 m (dip) crush pillar with a 4 m holing at a vertical depth of 600 m below surface.

The results from these modelled stress profiles were used to determine the average pillar stress (APS). The models were run elastically to determine the maximum stress predicted in the crush pillars, which also allows a comparison between the tributary area theory (TAT) and the numerical modelling methods.

Figure 25 presents a graph showing the stress profile across the crush pillar, as well as the APS determined using both the TAT and numerical modelling (FLAC3D) methods. The results indicate that the APS applying the TAT method is calculated to be ~391 MPa, compared to a value of ~366 MPa predicted by the numerical modelling. The profiles indicate that the pillar corners and sides are predicted to experience significantly higher stresses than the pillar core.

Applying a K-value of 58 MPa (based on the back analysis results), the factor of safety is calculated at ~0.3 (using the modelled APS), which is less than 1. Therefore the 3 m (width) × 4 m (length) pillars are predicted to crush.

Figure 26 presents a strain profile through the core of the crush pillar, indicating that the core of the 4 m crush pillar would experience strain levels of ~6.5 mm/m, compared to the sidewalls, which are predicted to experience strain levels of up to ~9.5 mm/m. The corners are predicted to experience strain levels of up to ~15 mm/m. The strain in the core of the 6 m long crush pillar with

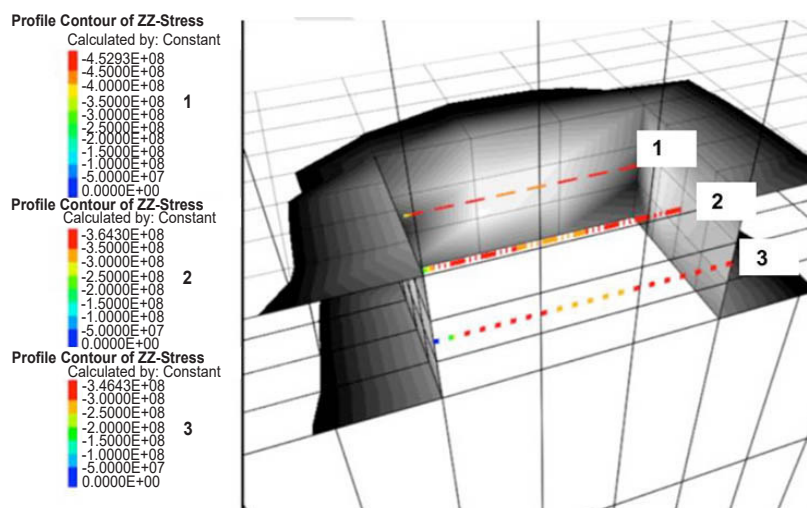


Figure 24—Schematic showing the locations of the vertical stress profiles across the crush pillar

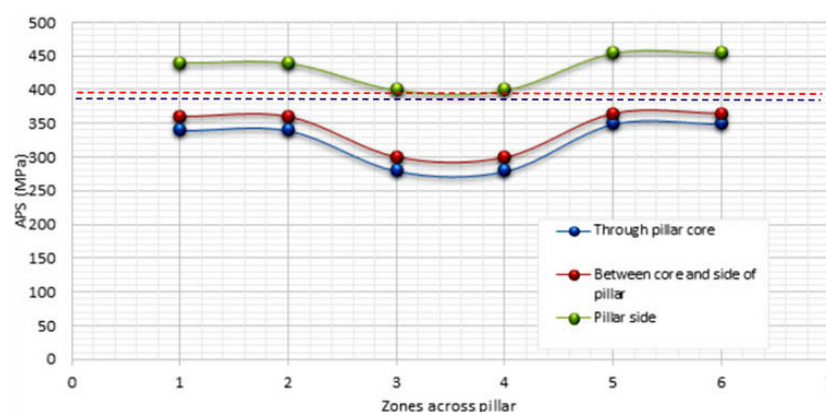


Figure 25—Schematic showing the profiles of vertical compressive stress in the pillar at different locations through the pillar (red indicate APS based on TAT; purple indicates APS based on FLAC3D)

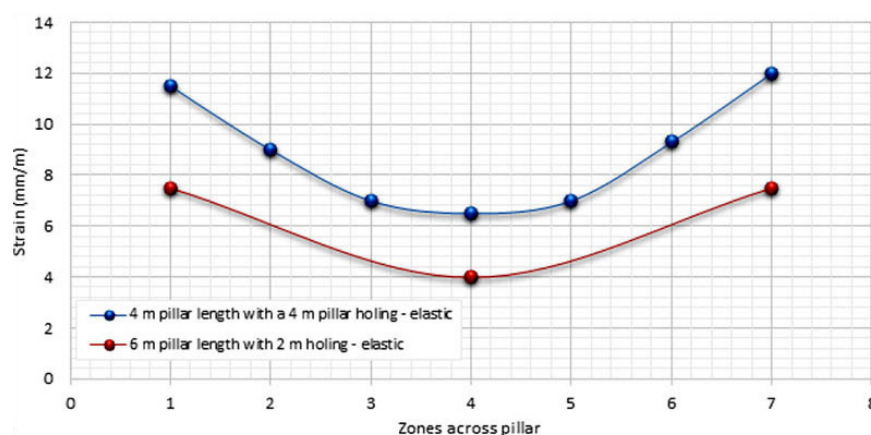


Figure 26—Schematic showing the vertical strain through the core of the crush pillar

a 2 m holing is ~4 mm/m, which is approximately a third less than the ~6.5 mm/m in the 4 m crush pillar with a 4 m holing. The strain criterion of 0.9 mm/m is however exceeded throughout the 4 m and 6 m long crush pillars, indicating that all the zones within the crush pillars will reach peak stress/strain.

Due to the high pillar stress resulting in a low factor of safety of ~0.3 and the high vertical strain of ~6.5 mm/m in the pillar core, significant fracturing of the crush pillars was predicted for a depth of 600 m below surface (photographs taken underground at a

depth of 500 m below surface support the prediction that fracturing will occur, which may lead to pillar scaling). The sidewalls and the corners, which are predicted to experience vertical strain ranging between ~10 and 15 mm/m will most definitely result in slabbing, which may lead to scaling. The holing width does not seem to affect the crush pillar performance but may impact roof stability between the pillars. The models predict that the stress overlap between the crush pillars will not result in the formation of a high tensile zone within the holing (analytical method suggests a tensile zone height

Application of an empirical method and numerical modelling to the Merensky Reef crush pillar stability

of less than 20 cm). However, experience has shown that a 2 m pillar holing at depths that may result in fracture formation in the roof, will tend towards a stable arch shape as a result of the compressive stress. The horizontal clamping stress from the adjacent crush pillars appears to be lower in a 4 m holing width, compared to a 2 m holing width. In addition, the potential for dilation along persistent reef-parallel stringers, is more likely to occur in a 4 m holing compared to a 2 m holing.

Summary

This paper presents a method for back analysis of rock mass strength, and pillar stress strain behaviour using advanced inelastic numerical modelling. The study further illustrates how the different variables impact the stability of crush pillars, including the pillar dimensions (width, height, and length), the dip of the orebody, and differences in layouts (such as holing widths and the presence of ASGs immediately adjacent to the crush pillars). Based on the findings, it is suggested that an empirical method be used as a first step due to its many limitations, rather than the final step in the pillar design process. This should then be followed by an in-depth analysis using advanced inelastic numerical modelling to investigate all possible failure mechanism.

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