MANDELA MINING PRECINCT
MINDS FOR MINES

TECHICAL SERVICES OPTIMISATION

Guidelines for Pillar design in Bord-and-Pillar operations

science & innovation
Department: Science and Innovation
REPUBLIC OF SOUTH AFRICA

CSIR

MINERALS COUNCIL SOUTH AFRICA

MANDELA MINING PRECINCT
MINDS FOR MINES

About the Mandela Mining Precinct

The Mandela Mining Precinct is a Public-Private Partnership between the Department of Science and Innovation and the Minerals Council South Africa. The Precinct is jointly hosted by the Council for Scientific and Industrial Research and the Minerals Council. The Mandela Mining Precinct is an initiative aimed at revitalising mining research, development and innovation in South Africa to ensure the sustainability of the industry. This is achieved through the South African Mining Extraction, Research, Development and Innovation (SAMERDI) strategy. The strategy comprises six research programmes:

1. Longevity of Current Mining;
2. Mechanised Mining Systems;
3. Advanced Orebody Knowledge;
4. Real-Time Information Management Systems;
5. Successful Application of Technologies Centred Around People; and
6. Test Mine.

This guideline was developed under the Mechanised Mining Systems research programme. The programme is aimed at providing sustainable mechanised drill, blast and mechanical rock breaking solutions in advancement towards atomised systems to facilitate achieving zero harm, whilst maintaining and defending desired production rates at minimised costs, within the au and PGM mining industries.

ISBN: 978-0-7988-5651-5

Contact details enquiries@mandelaminingprecinct.org.za
www.mandelaminingprecinct.org.za

Editors: Martin Pretorius (Mandela Mining Precinct) and Francois Malan (University of Pretoria Enterprises)

Contributing authors: Dr Dave Roberts (Council for Scientific and Industrial Research
Prof Johann Esterhuyse (University of Pretoria Enterprises)

External Review: Roy Murley, Glencore; and Alex Fenn, Sibanye Stillwater

Executive Summary

Research was commissioned by the Mandela Mining Precinct to review the rock engineering principles that may affect the planned mechanisation of South African tabular mines. The key aspects considered were larger pillar sizes and lower extraction ratios at increasing depths for bord-and-pillar layouts; the principles of support design for mechanised mining; a possible change in seismic behaviour in deep mines if the operations are mechanised; and the use of numerical models for pillar design. Based on the information collected, a lack of knowledge on hard rock pillar strength was deemed the most pressing issue to address. Through this guideline, the authors posit that improved pillar strength formulae are required to optimise bord-and-pillar layouts at greater depths.

The authors were unable to find any readily available analytical solution in the literature consults that describes the effect of depth on the extraction ratio for a bord-and-pillar layout. A suitable solution was derived for this project and the results indicate that the maximum depth at which an acceptable extraction ratio can be achieved is strongly dependent on the K-value used in the pillar strength formula. The uncertainty regarding appropriate formulae and K-values makes an estimation of viable extraction ratios at depths that are increasingly difficult. This is a significant hurdle in terms of the future mechanisation of the South African mining industry. Successful mechanisation may also be impacted if there is a very weak contact (parting plane or clay layer) present in the pillars. The pillar strength may be compromised to such an extent that normal bord-and-pillar layouts with an acceptable extraction ratio may not be feasible. Further research may be required to determine the pillar strength in areas where weak layers are present.

The important modelling tools available for simulating pillar stress and strength can be grouped into boundary element and finite difference approaches. Boundary element models are useful when computing the closure and stress distribution across large-scale layouts. Finite difference codes are useful to simulate the failure mechanisms in a single pillar. This report illustrates the important applications of both tools. The TEXAN code with the limit equilibrium model can accurately simulate pillar stress and pillar failure in a large-scale model containing many pillars. In contrast, the FLAC code is useful to simulate pillar failure mechanisms, such as the effect of a weak layer on pillar strength.

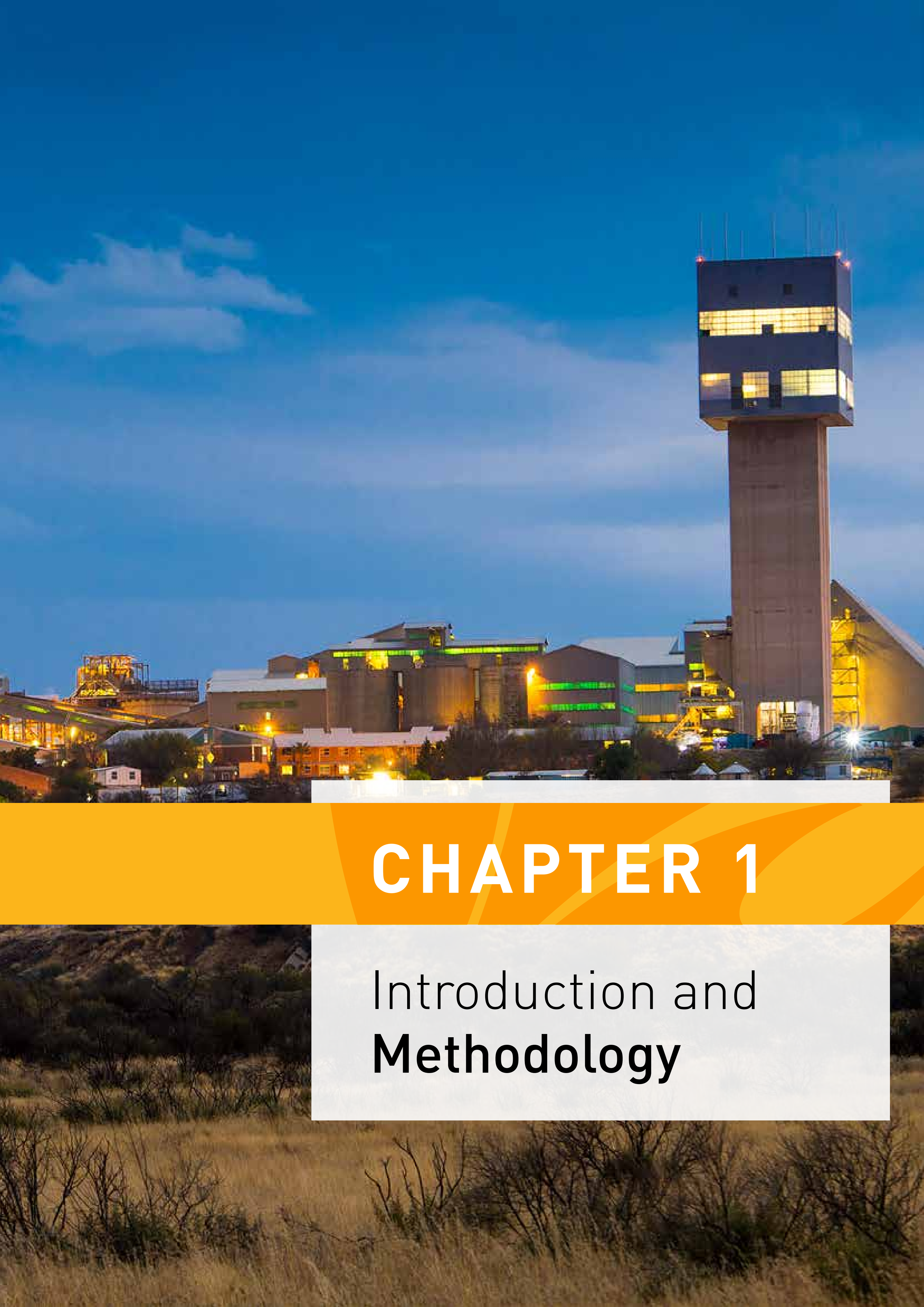
The authors deemed it necessary to obtain up-to-date information from industry on the methodologies used for pillar design. A workshop with rock engineering practitioners was held in October 2020 and a follow-up questionnaire was sent to industry to determine the latest industry best practice in terms of pillar design. From the data collected, it is noteworthy that most of the operations still use the empirical Hedley and Grant formula. Numerical modelling is currently only used to a limited extent to optimise the designs.

Guidelines for hard rock pillar design in South African bord-and-pillar operations are presented in this report as well as a proposed multi-year research project to address the unknowns in terms of pillar design.

Table of Contents

List of Figures

List of Tables



CHAPTER 1

Introduction and **Methodology**

1.1 INTRODUCTION

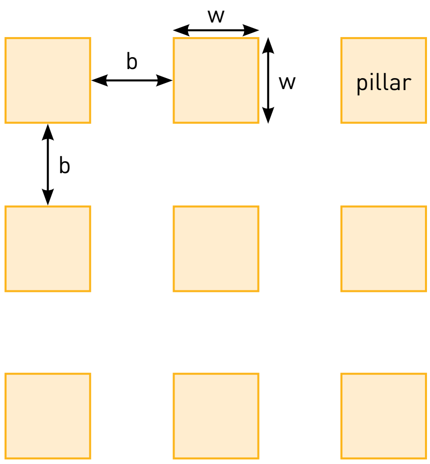
This report focusses on pillar strength for bord-and-pillar designs. Some of the relevant information from the previous reports is included herein to allow for a holistic, standalone document. Additional information on numerical modelling and the effect of different exponents in the power-law pillar strength formula is also included. This guideline was developed on the back of a review of rock engineering principles that may affect the mechanisation of South African tabular mines. It provides preliminary guidelines for rock engineering best practice in terms of pillar design. It should be emphasised that a key finding of the study was that there are still significant gaps in terms of rock engineering knowledge in this area. This presents a major hurdle in terms of the mechanisation of layouts and therefore, only preliminary guidelines can be provided.

The methodology followed in developing this guideline is described in the illustration below:

An analytical solution of extraction ratio as a function of depth

The shallow orebodies typical of platinum and chrome mines in the Eastern Bushveld Complex illustrated that mechanisation is feasible using a bord-and-pillar layout. However, based on the commonly accepted pillar strength formulae, the pillar sizes increase substantially with depth and the extraction ratio therefore becomes too small at depth. This concept is explored in more detail below and is of particular significance as it presents a major hurdle to future mechanisation.

No analytical solution to describe the effect of depth on the extraction ratio for a bord-and-pillar layout is readily available in the literature reviewed. It was therefore newly derived for this project and the details thereof are provided below. Consider the bord-and-pillar layout shown in Figure 1, located at a depth of H. The dip is considered to be 0° and the extent of the layout in the two lateral directions is considered to be very large. Tributary area theory (TAT) can therefore be used as a good approximation to calculate the stresses acting on the pillars (Ryder and Jager, 2002).



$$e = \frac{(b + w)^2 - w^2}{(b + w)^2}$$

Figure 1. Idealised bord-and-pillar layout.

For the layout shown above, the average pillar stress (APS) is given by (Ryder and Jager, 2002):

$$APS = \frac{\rho g H}{1 - e} \qquad (1)$$

where ρ is the density of the rock, g is the gravitational acceleration, and e is the extraction ratio. Equation (1) can be written as follows to give the extraction ratio:

$$e = 1 - \frac{\rho g H}{APS} \qquad (2)$$

For the bord-and-pillar layouts, the pillars are designed to remain stable and the APS needs to be limited to prevent failure of the pillars. Typically, a factor of safety (Fs) is used in the design and this is given by:

$$F_s = \frac{\delta s}{APS} \qquad (3)$$

where δs is the pillar strength. This can be written as:

$$APS = \frac{\delta s}{F_s} \qquad (4)$$

Equation (4) can be inserted in equation (2) to give:

$$e = 1 - \frac{\rho g H F s}{\delta s} \qquad (5)$$

Typically, a power-law strength formula is used to determine the pillar strength, δs. More details are provided below. In South African mines, the use of the Hedley and Grant strength formula is ubiquitous (Malan and Napier, 2011) and it is given by the formula:

$$\delta_s = \mathrm{K} \frac{w^{0.5}}{h^{0.75}} \qquad (6)$$

where K is the strength of the rock mass in the pillar, w is the pillar width, and h is the pillar height (mining height). Equation (6) can be inserted in (5) to give:

$$e = 1 - \frac{\rho g H F_S h^{0.75}}{K w^{0.5}} \qquad (7)$$

By assuming that

$$\gamma = \frac{\rho g F_S h^{0.75}}{K w^{0.5}} \qquad (8)$$

Equation (7) can be simplified as:

$$e = 1 - \gamma H \qquad (9)$$

The extraction ratio, e, is therefore a simple decreasing linear function of the depth, H, and the rate of decrease is dependent on the modifier, γ. The parameter, γ, is a function of the assumed pillar strength and the factor of safety.

Equation (9) is plotted in Figure 2 for a typical case study in a platinum mine. The factor of safety on the pillars is maintained at a value of 1.6, g = 9.81 m/s2. The mining height (pillar height) is assumed to be 2 m and the pillar width is maintained at 6 m to give a width to height ratio of 3:1. Three different values of rock mass strength (K) are used in the diagram to illustrate the effect of the extraction ratio. There is currently significant debate in the industry regarding the appropriate value of K to use and this is explored later in this guideline.

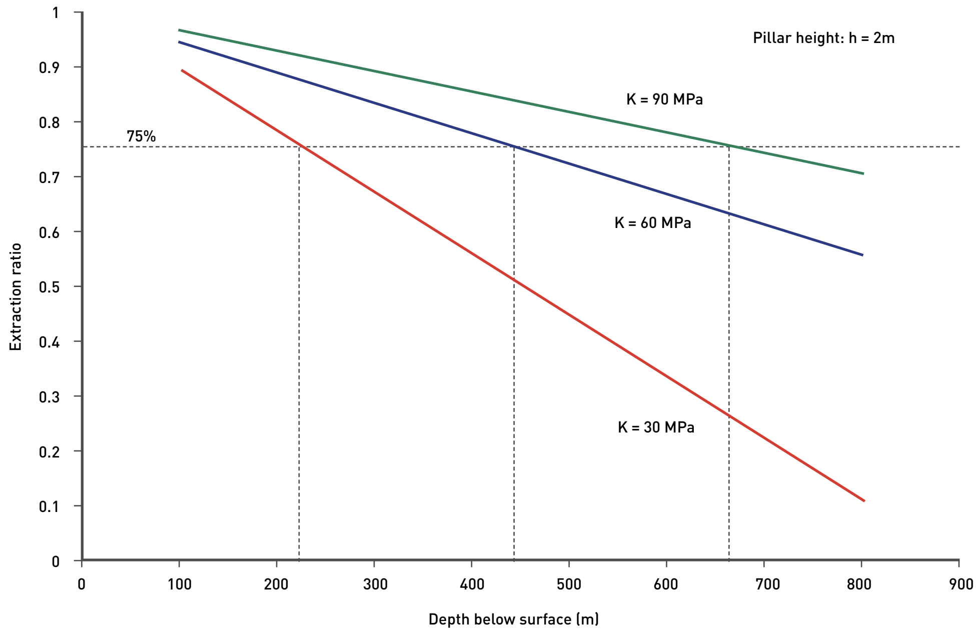


Figure 2. Decrease in extraction ratio with an increase in depth (pillar height = 2 m)

It is indicated in Figure 2 that the maximum depth at which an extraction ratio of 75 % can be achieved is dependent on the K value used in the pillar strength formula. For K = 30 MPa, the maximum depth for this extraction ratio is only 220 m, whereas it increases to 660 m for K = 90 MPa. Determining the actual pillar strength is therefore critical.

A parameter also playing a strong role in the extraction ratio is the pillar height. This is illustrated in Figure 3 where a pillar height of 1.5 m is used and this increases the pillar width to height ratio to 4:1. The results can be compared with Figure 2 for a pillar height of 2 m. Note that for K = 90 MPa, the depth at which a 75% extraction ratio can now be achieved has increased from 660 m (2 m height) to 820 m (1.5 m height). In many mines, a significant portion of waste is mined as part of the cut, and the extraction ratio can be easily improved by reducing the mining height. In terms of mechanisation, the use of XLP equipment therefore needs to be investigated.

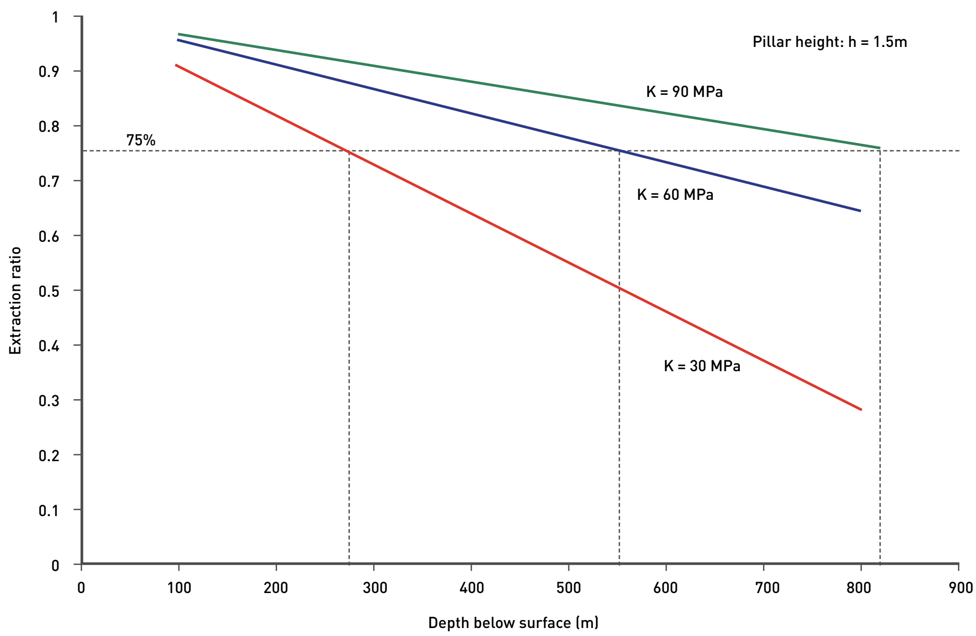


Figure 3. Decrease in extraction ratio with an increase in depth (pillar height = 1.5 m).

The value of expressing the extraction ratio in the form of an equation (5) is that it clearly illustrates the dependence of the extraction ratio on the pillar strength. The higher the pillar strength, the greater the extraction ratio that can be maintained with increasing depth. The difficulty associated with estimating pillar strength is explored in the next section.

1.2 ESTIMATING PILLAR STRENGTH

An extensive overview of the problems associated with pillar designs in the Bushveld Complex is given in Malan and Napier (2011). A summary is given below to highlight the problems associated with the commonly used Hedley and Grant pillar strength formula (assumed in the section above).

Empirical power-law strength formulae are a popular and standard method to determine pillar strength in the South African mining industry. The typical form of the formula is well known, but it is repeated below for completeness:

$$\delta_s = \mathrm{K}\,\frac{w^{\alpha}}{h^{\beta}} \qquad (10)$$

where K reflects the fitted 'strength' of the in-situ rock, w is the width of the (square) pillar, and h is the height in metres. The parameters α and β are equal to 0.46 and 0.66 respectively, in the well-known Salamon and Munro (1967) coal pillar strength formula. For the Hedley and Grant (1972) formula used in the Bushveld Complex, α = 0.5 and β = 0.75. The K-value is typically taken between a third and two-thirds of the laboratory UCS strength of the rock.

The pillar volume is given by $V = w^2h$. Defining the width to height ratio, $R = w/h$, equation (10) can be expressed in the alternative form:

$$\delta_s = KV^{(\alpha-\beta)/3}R^{(\alpha+\beta)/3} \qquad (11)$$

It can be seen from equation (11) that if $\alpha = \beta$, the pillar strength is independent of the pillar volume whereas if $\alpha > \beta$, as in the Salamon and Munro; and Hedley and Grant formulations, the pillar strength is predicted to decrease as the pillar volume is increased even if the pillar shape is unchanged. An objection raised by Bieniawski (1992) is that, according to the power law formulation, the cube strength (w = h) would continue to decrease indefinitely with side length. This is considered unreasonable (Hustrulid, 1976). Most laboratory and field data indicates that the w:h strengthening curve has a zero or positively upward curvature (Ryder and Jager, 2002). The power law formula predicts the downward curvature in contrast. An alternative 'linear' equation, with no volumetric size effect, was therefore proposed and it directly expresses the strengthening effect of the w:h ratio (Bieniawski, 1992).

$$\delta_s = K\left(A + B\,\tfrac{w}{h}\right) \qquad (12)$$

where K (MPa) is the "in-situ" strength of a large block (w:h = 1) of pillar material and A and B are dimensionless strengthening parameters such that A+B = 1. Ryder and Jager (2002) state:

"The power law and its derivatives are perhaps too entrenched in coal engineering to warrant withdrawing from them at this time, but in hard rock engineering, the simpler and probably more realistic linear forms are advocated for general use."

As many consultancy reports have been devoted to "tweaking" the Hedley and Grant formula, it is worthwhile to consider the original assumptions made when this formula that was derived in Canada. The uranium mines in the Elliot Lake area used a stope and pillar layout to mine the orebody. Narrow pillars about 76 m long were left along the dip. The pillars, which were chosen for analysis, were typically 3-6 m wide and 2.5–6 m high. The width to height ratio of most of the pillars was close to 1 and only a very few (3 in the database) had a width to height ratio of 2.5. This original formulation was derived for slender rib pillars and it can be questioned whether it is applicable to square pillars in South African mines with a width to height ratio greater than 2.5. From the original data used by Hedley and Grant, it is obvious that the dataset used was very small (28 pillars). This should be compared to the coal database of Salamon and Munro (1967), which included 125 pillars of which 27 were collapsed. The width to height ratio of the failed uranium pillars varied from 1.1 to 1.5. Only 3 of these pillars were 'crushed' and 2 were 'partially crushed'. Of further concern is that it is stated in the Hedley and Grant paper that: *"The information on complete pillar crushing was obtained second-hand because it happened in mines which are closed."*

This work was conducted prior to the use of computer-based numerical modelling to determine pillar stress; and the tributary area theory was used. The first step was to adopt the power law strength formulation given in equation (10). Hedley and Grant acknowledge that this equation refers to square pillars, whereas those in the uranium mines were long and narrow. Their assumption rested on the strength of the slender pillars which will not be much greater than a square pillar of width equalling the minimum width of the long pillar. Secondly, from the extrapolation of laboratory tests, it was estimated that the value of K is 26 000 psi for a 1-ft cube. Thirdly, appropriate values for parameters α and β had to be derived. Three different sets of values were available to them at that stage in the literature. The value for α was relatively constant at 0.5 and therefore, Hedley and Grant also adopted this value. As β varied more, a new value was computed and their approach was to focus on the three failed pillars in the database. For each of these pillars, the tributary area stress in the table was assumed to be the pillar strength. This value, as well as the K-value and α = 0.5, were substituted into Equation [1] and the value of β was solved for each pillar. The calculated values of β ranged from 0.736 to 0.768 with a mean of 0.75. This value was adopted and it resulted in the now familiar Hedley and Grant formulation. Clearly the formulation above is based on a large number of assumptions, and the applicability of this formulation to the design of hard-rock pillars in the Bushveld Complex in South Africa becomes highly questionable.

A significant effort to develop a new pillar strength formula for the Bushveld Complex was undertaken by the PlatMine research programme. Two references were available to the authors, namely, the 2005 PlatMine report and a recent draft paper by Watson et al. (2020). For the UG2 study, pillar data was collected from three mines near Thabazimbi. The new equation was therefore derived for the UG2 in the Western Bushveld. The database consisted of 167 pillars of which 134 were intact and 33 were failed. Most of the pillar w:h ratios ranged between 1.5 and 4, with the largest proportion being between 2.0 and 3.0. Pillar loads were estimated using elastic MinSim and MINF (Spottiswoode and Milev, 2002) modelling. It is acknowledged in the 2005 report that: "A major stumbling block throughout has been the difficulty of establishing realistic pillar loads through MINSIM/MINF modelling of the complex Bushveld layouts involved." The preliminary estimates in the report indicated typical strengths in the range 140 to 200 MPa for UG2 pillars of crush pillar size. The maximum likelihood analysis was used to estimate the best fit for the parameters given in equation (10). From this study, the PlatMine UG2 equation was derived as:

$$\delta_{UG2} = 67\frac{w^{0.67}}{h^{0.32}}\ (MPa) \qquad (13)$$

As $\alpha > \beta$ in this equation, it predicts a different behaviour compared to Hedley and Grant namely that for a constant w:h ratio, the pillar strength increases as the pillar volume increases. This formula predicts significantly higher pillar strengths than the classical methodology. For example, consider a 7 m x 7 m pillar at a stoping width of 2.5 m. The PlatMine formula predicts a strength of 184 MPa, while the Hedley and Grant formula only gives a strength of 39.9 MPa for an assumed value of K = 30 MPa (Spencer, 2009). This is a huge difference in values and care should therefore be exercised with this new formula. Watson et al. (2020) also note that: "These formulae may be used cautiously on all Bushveld platinum mines with similar geotechnical and geomechanical conditions to the pillars in the database."

A similar strength formula was derived for the Merensky Reef pillars and this is given by (Watson et al., 2008):

$$\delta_{Merensky} = 86\,\frac{w^{0.76}}{h^{0.36}}\ (MPa) \qquad (14)$$

Again note that $\alpha > \beta$ in this equation. Figure 4 illustrates the difference in pillar behaviour for different values of the exponents in the power-law formulation (Malan and Napier, 2020). It is not clear if either of these formulations predict the correct pillar behaviour.

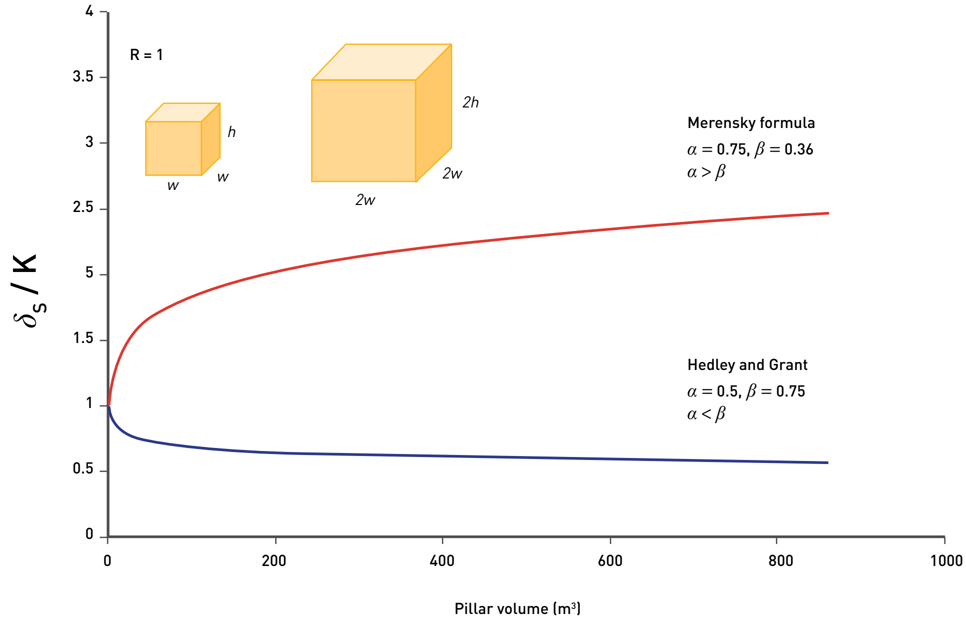


Figure 4. Effect of pillar volume on pillar strength (Malan and Napier, 2020).

The new UG2 and Merensky Reef pillar formulae predict pillar strength substantially higher than those used in the historical bord and pillar layout designs. This is illustrated in Figure 5.

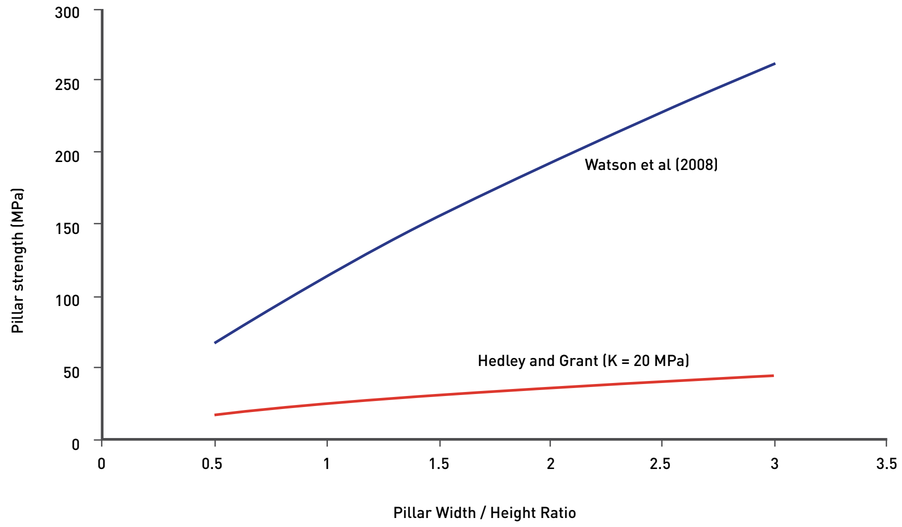


Figure 5. A comparison of the Merensky pillar strength predicted by the new Watson formula and the traditional Hedley and Grant using a K-value of 30 MPa (a third of the average UCS obtained from laboratory tests of Merensky Reef) (Malan and Napier, 2011).

Martin and Maybee (2000) provide a good overview of the different empirical strength formulae that were developed to predict pillar strength. A comparison of these formulae is given in Figure 6. These curves were calculated for a pillar height of 5 m. The pillar strength is normalised to the UCS of the rock. Esterhuizen (2006) conducted an evaluation of the strength of slender pillars. Figure 7 illustrates the published case histories of failed pillars from hard-rock metal mines. This figure illustrates that the pillar strength becomes highly variable as the width to height ratio decreases. For a ratio of 1, the pillar strength is not expected to exceed 65% of the laboratory UCS strength (but it can also be significantly lower than this value). Both these graphs, based on many empirical studies, indicate that the pillar strength typically does not exceed 70% of the laboratory UCS up to a pillar w:h ratio of 2-2.5. The 7 m x 7 m pillar strength of 184 MPa predicted by the PlatMine UG2 formula for a stoping width of 2.5 m (w:h = 2.8) appears to be anomalous and further testing is required. From the available data, Lousteau (2016) estimated a UCS value for the UG2 at Booysensdal between 90 MPa to 110 MPa while UG2 laboratory tests on specimens from a neighbouring mine provided values of typically 125 MPa to 150 MPa. This would possibly indicate a lower strength for the small w:h ratio pillars compared to that predicted by the PlatMine formula. This apparent contradiction between the PlatMine formula and previously published empirical data by other authors needs to be investigated. For completeness, Figure 8 is included to illustrate some of the alternative empirical pillar strength formulae used in the coal industry.

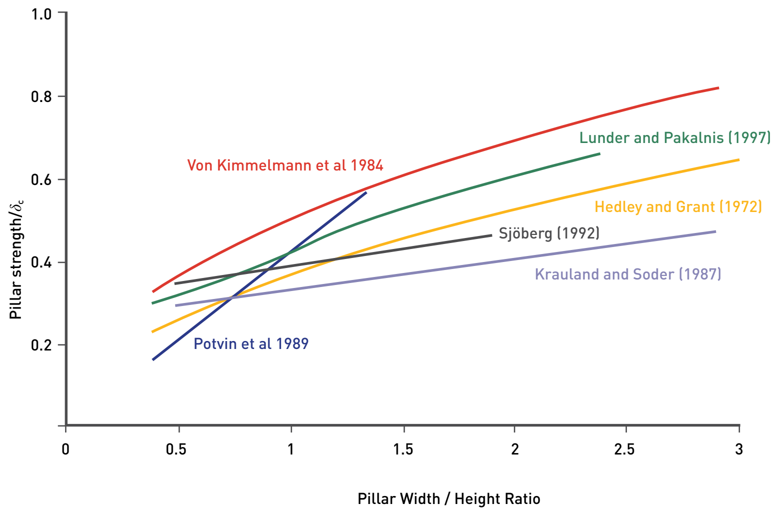


Figure 6. Comparison of empirical pillar strength formulae (Martin and Maybee, 2000).

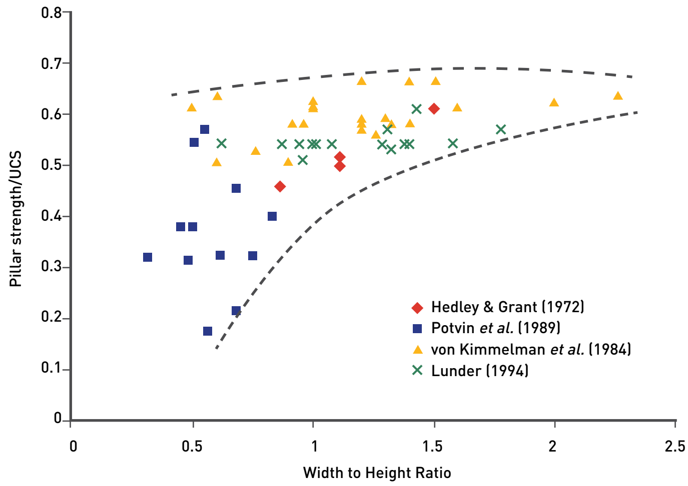


Figure 7. Pillar stability graph showing examples of failed pillars from hard-rock mines (Esterhuizen, 2006).

SI NO.	AUTHOR	EQUATION	APPROACH
1.	Obert and Duval (1967)	$S = k\left(0.78 + 0.22\frac{w}{h}\right)$	Lab test of in situ size
2.	Salamon and Munro (1967)	$S = 7.2\frac{w^{0.46}}{h^{0.66}}$	Constant strength from pillar cases
3.	Bieniawski (1975)	$S = k\left(0.64 + 0.36\frac{w}{h}\right)$	In situ size strength considered
4.	Sherory (1992)	$S = 0.27kh^{-0.36} + \left(\frac{H^{0.46}}{250} + 1\right)\left(\frac{w}{h} = 1\right)$	Theoretical, empirical method and case studies
5.	van der Merwe and Malhey (2013)	$S = 5.47\,\frac{w^{0.8}}{h}$	Case studies using overlap technique Constant strength

Where: S=pillar strength (MPa), k=compressive strength of coal sample, w=width of square pillars or smaller width of rectangular pillars (m), h=extraction heigth (m), and H=depth of cover (m).

Figure 8. Alternative empirical coal pillar strength formulae (Kumar and Sing, 2018).

In conclusion, Salamon and Munro (1967) summarised the risks of empirical equations well when they wrote in their paper: *"The work described in this paper is essentially empirical, and the results, therefore, should not be extrapolated beyond the range of the data which were used to derive them."*

Based on the literature survey presented in this section, it is clear that there is still significant uncertainty regarding pillar strength and appropriate formulae to use when designing bord-and-pillar layouts. This is a significant hurdle in terms of future mechanisation of South African mines at greater depths. It is recommended that experimental work be funded to obtain better estimates of these pillar strengths and to agree on the most appropriate strength formulae to use.

1.3 EFFECT OF WEAK LAYERS IN PILLARS

A further rock engineering problem that may impact mechanisation is when there is a very weak contact (parting plane or clay layer) present in the pillars. The effect of these weak layers is described in Malan and Napier (2011). These layers weaken the pillars substantially and three major mine collapses have been reported in recent years in platinum and chrome mines in Southern Africa.

Spencer (1999) reported on the collapse of the pillars and the subsequent closure of the Wonderkop Chrome Mine (close to Rustenburg) in 1998. The mine exploited the lower group chromite seams, namely, the LG6 and LG6a. The pillar design was done using the Hedley and Grant pillar strength formula. The pillar sizes were 12 m x 6 m. K was assumed to be a third of the laboratory strength of the rock, namely, 27.3 MPa. The stoping width was 2 m, so the w:h ratio was at least 3 if the smallest dimension of the pillars is considered. The strength of the pillars was calculated to be 45.9 MPa. The proximity of the mine to the Spruitfontein dome has influenced the structure of the LG6 and LG6a seams and this has resulted in thick clay layers (up to 300 mm in some places) traversing the pillars in some areas (Figure 9). The position and thickness of this weak layer is highly variable.

Figure 9. Presence of weak clay layers in the pillars at Wonderkop mine. This photograph illustrates the presence of a clay layer between the LG6a chromitite and the pyroxenite below it (Malan and Napier, 2011).

Figure 10. Mode of pillar failure where a clay layer was found between the upper LG6A chrome and the pyroxenite below it. This slippery layer facilitates the fracturing of pyroxenite, causing it to scale out (left). The failures led to significant convergence, as can be seen on the photograph on the right.

In 1997, some joints were beginning to open at the corners and sides of some of the pillars. Following these observations, a system of barrier pillars were introduced with a width-to-height ratio of at least 10. The condition of the pillars nevertheless continued to deteriorate. To reinforce the pillars along the main dip belt and road decline, two strategies were adopted, namely, waste stowing between the pillars and mesh and lacing of the pillars. During April 1998, the failure process accelerated and the rate of closure in some areas increased to 1.8 mm/day. Numerous falls of ground occurred and the management decided to cease operations at this stage. A back analysis of this pillar failure was conducted by Malan and Napier (2011) and the Hedley and Grant formulation was used to back-calculate the K-value for the pillars. A value as low as 6 MPa was obtained.

Another large-scale pillar collapse occurred in the Eastern Bushveld Complex at Everest Mine. At this mine, a clay layer is also present at the hanging wall/pillar contact (Figure 11). The reef exploited in this area is the UG2. The original mine design was conducted using the Hedley and Grant pillar strength formulation with a conservative K-value of 35 MPa. The mining height was 2 m. In mid-2008, some concern was expressed regarding the stability of the pillars and a minor collapse occurred during this time. In an attempt to reinforce some of the pillars, many were supported using fibre reinforced shotcrete. This did not stop the deterioration, however, as shown in Figure 12 with the cracked shotcrete clearly visible. During December 2008, operations were suspended at the mine when the decline was affected by instability.

Figure 11. Weak clay layer (beige colour) present at the contact between the pillars and the hanging wall at Everest Mine.

Figure 12. Pillar failure at Everest Mine after attempts to strengthen the pillars with shotcrete (Malan and Napier, 2011).

From these studies, the drawbacks of using empirical pillar strength formulas are obvious. The mechanisms of failure in all three large-scale collapses were caused by the presence of layers of very low friction angle present in the pillars. This substantially weakened the pillars. The empirical formulae were developed for different rock types and the application of these formulas outside the limits for which they were developed resulted in the large mine collapses described here.

An aspect to highlight is that mechanisation may not be possible in areas where weak layers are present in the pillars. The pillar strength seems to be compromised to such an extent that normal bord and pillar layouts with an acceptable extraction ratio may not be feasible. One solution proposed for these ground conditions is hybrid mining where panels are mined conventionally. This is not deemed an elegant solution, however, in the drive towards much greater mechanisation of the mines. Further research needs to be conducted to determine the pillar strength in areas where the weak layers are present. There is currently no acceptable methodology to do this. Numerical modelling is an option, but calibration of the complex inelastic models is a significant hurdle to overcome at this stage.

1.4 NUMERICAL MODELLING AND PILLAR DESIGN

From the sections above, it appears that there is significant uncertainty regarding the current empirical formulae used in hard rock mines and additional work is required. The orebodies in South Africa are very different to those from which the Hedley and Grant formulation was derived. The restricted range of slender w:h ratios used when deriving the equation needs to be considered and selecting an appropriate K-value is difficult. An alternative to the empirical approach is to use numerical modelling, with appropriate failure criteria to determine the pillar strength. A vast amount of literature is available on attempts to simulate pillar failure. In the Milestone 4.3.1.1 report, a literature survey on some of the relevant material was provided. Some additional key aspects related to the modelling of pillars are considered below (also see Malan and Napier, 2011).

Day and Godden (2000) described the design of pillars on Lonmin's platinum mines. They used extensive underground surveys and computer back analysis studies to indicate the validity of their approach. Approximately 300 pillars per month were cut at Lonmin at that stage and the stable behaviour of the pillars served as further validation of the approach. They state that the methodology is applicable up to width to height ratio of 5.5, but not at greater values owing to the onset of squat pillar effects. This is surprising as the expectation is that a numerical method with an appropriate constitutive model will take care of the onset of squat pillar behavior by default. Pillar strength was estimated by two-dimensional FLAC modeling using the following Hoek and Brown failure criterion:

$$\delta_1 = \delta_3 + (\mathrm{m}\delta_c\delta_3 + s\delta_c^{\,2})^{0.5} \qquad (15)$$

where δ_c = uniaxial compressive strength (MPa) and m and s are constants related to the properties of the rock. The constant m was determined from laboratory testing and s equals to 1 for intact specimens. In situ values for the constants m and s were derived by application of rock mass quality ratings such as RMR using the equations of Priest and Brown (1983) for undisturbed rock masses:

$$m = m_i e^{\frac{RMR-100}{28}} \qquad (16)$$

$$s = e^{\frac{RMR-100}{9}} \qquad (17)$$

The authors derived in situ values for m and s for the UG2 and Merensky Reefs at Lonmin Mine. Typical values used in the modeling are as follows: UG2; m = 25.83, s = 0.51 for a RMR of 94, Merensky; m = 8.7, s = 0.57. The resulting simulated pillar strengths seem plausible when the pillar width to height ratio is low.

It should be noted that this modelling approach retains elements of a "semi- empirical" approach. Equations (16) and (17) may not be correct not for the UG2 and Merensky pillar material. When conducting pillar design, it should be considered that numerical modelling methods are not always superior to empirical derived equations. Mostyn and Douglas (2000) made the following statement regarding this failure criterion for intact rock.

"…there are inadequacies in the Hoek-Brown empirical failure criterion as currently proposed for intact rock and, by inference, as extended to rock mass strength. The parameter mi can be misleading, as mi does not appear to be related to the rock type. The Hoek-Brown criterion can be generalized by allowing the exponent to vary. This change results in a better model of the experimental data."

Martin and Maybee (2000) investigated the strength of hard rock pillars by using both empirical pillar strength formulae and numerical modeling using a Hoek-Brown failure envelope. They reached the conclusion that two-dimensional finite element analyses using conventional Hoek and Brown parameters for typical hard rock pillars predicted rib pillar failure envelopes that did not agree with the empirical pillar failure envelopes. The conventional Hoek-Brown failure envelopes over-predict the strength of hard rock pillars. Hoek et al. (1995) described the rock mass conditions for which the Hoek-Brown failure criterion can be used. The criterion is only strictly applicable to intact rock or heavily jointed rock masses that can be considered homogenous and isotropic. For cases in which rock mass behaviour is controlled by a single discontinuity or joint set, a criterion that describes the shear strength of the joints should rather be used. Such an example is shown in Figure 13. The implication of this is that for the three case studies of pillar failure discussed above, an explicit simulation of the clay layer will probably be required.

Figure 13. An example of a pillar which contains a prominent joint dipping at almost 45°. This joint will have to be modelled explicitly when using a numerical modelling code (Malan and Napier, 2011).

An example of using the FLAC code to simulate the behaviour of pillars with weak clay layers is given in Malan and Napier (2011). The pillar composition simulated was the LG6/LG6A package shown further in the report in Figure 18. The qualitative effect of a strong pyroxenite layer within a chromitite pillar was modelled in 2D. The weak contacts, including the weak hanging wall and footwall contacts were built into the model. In situ strain-softening parameters from studies carried out in the Bushveld Complex were used. The hanging wall and footwall were assigned the same properties as the pyroxenite layer. Symmetry was assumed for both the vertical and horizontal centerlines in the following layout in the FLAC finite difference code. The grid size was 0.1 m × 0.1 m. The complete stress-strain curves could be modelled. The presence of a weak interface between the layer and the body of the pillar had virtually no effect, even if the friction angle of interface 1 was set as low as 6°. In contrast, low friction angles on the hanging wall contact (interface 2) had a powerful effect, reducing the peak strength p of the pillar by allowing lateral deformation and reduction of confinement, and reducing also the residual strength. This type of modelling work will have to be extended for the proposed future project described below.

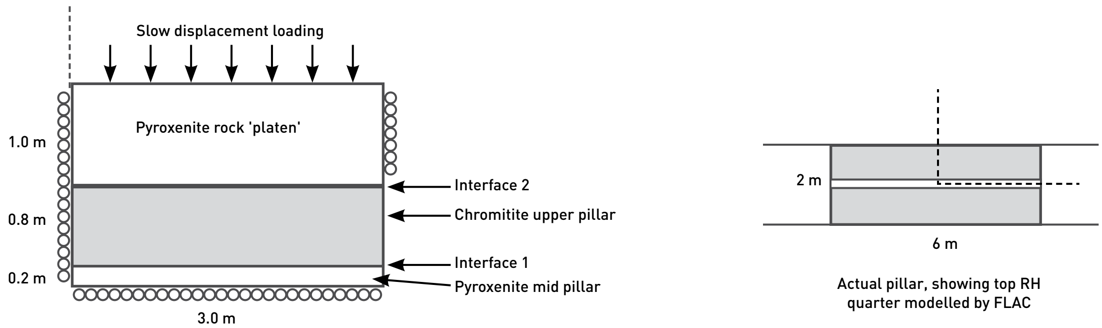


Figure 14. FLAC model to simulate the effect of weak interfaces in the pillar (Malan and Napier, 2011).

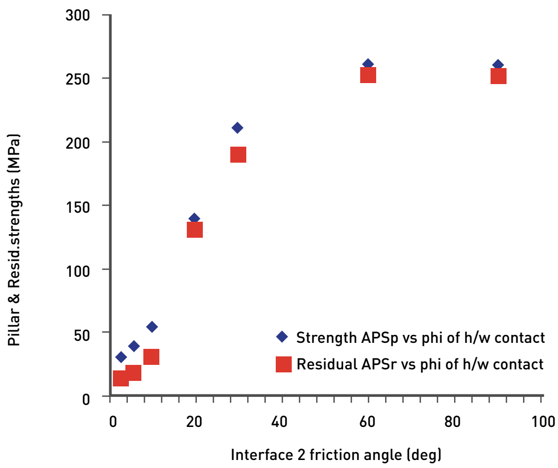


Figure 15. Peak and residual pillar strength versus hanging wall contact friction angle (Malan and Napier, 2011).

A recent important development was the implementation of a time-dependent limit equilibrium model in the TEXAN boundary element code (Napier and Malan, 2018). It can simulate on-reef failure, so is therefore ideal to simulate pillar failure in bord and pillar layouts. It gives a representation of the on-reef horizontal and vertical stress distribution adjacent to a pillar edge as shown in Figure 16. The model assumes that if the reef material fails, a relationship exists between the reef normal and parallel stress in the seam ahead of the face. The reef-parallel stress $\delta_s\ (x,t)$ at position x and time t is balanced by a frictional shear traction $\mu_I \delta_n\,(x,t)$ at the interfaces between the fractured material and the intact reef. The parameter μ_I is the interface friction coefficient and $\delta_n\,(x,t)$ is the traction component normal to the reef. The limit strength model specifies that a relationship exists between $\delta_n\,(x,t)$ and $\delta_s\,(x,t)$. This is specified as:

$$\delta_n\,(x,t) = \delta_c\,(x,t) + m\,(x,t)\;\delta_s\,(x,t) \qquad (18)$$

The strength envelope parameters $\delta_c\,(x,t)$ and $m\,(x,t)$ are functions of position and time. This allows the time-dependent failure of the reef material to be studied. From Equation 10 and the model shown in Figure 16, the average reef-parallel confining stress $\delta_s\,(x,t)$ at position x and time t can be given by the differential equation of the form:

$$H\;\frac{\partial \delta_s\left(x,t\right)}{\partial x} = 2\mu_I\,[\delta_c(\mathrm{x,t}) + m\,(x,t)\;\delta_s\,(x,t)] \qquad (19)$$

The implication of this model is that the stress increases exponentially towards the edge of the fracture zone. A drawback of the model is that the failure is restricted to the plane of the reef. It is nevertheless an elegant model to simulate pillar failure on the reef horizon in a large-scale model containing many pillars.

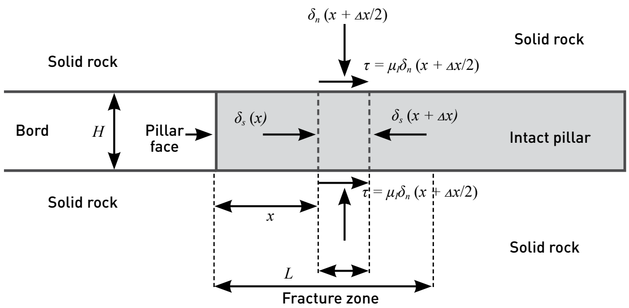


Figure 16. Force balance for the average reef-parallel confining stress and reef-normal stress at position x and time t in a plane section at right angles to the mining face. The stoping width is H and the distance of the fracture zone edge from the pillar face is L.

Figure 17. Simulated failure in a pillar reduction experiment simulated using the TEXAN code and the limit equilibrium model (Napier and Malan, 2020). The red dots indicate the failed elements. The photograph on the right illustrate the condition of pillar B.

1.5 INFORMATION COLLECTED AT THE WORKSHOP AND FROM THE QUESTIONNAIRE

At the workshop, a summary of the information given above was presented to the rock engineering practitioners from industry. The detailed information collected at the workshop and the minutes is given in the Milestone 4.3.1.2 report. The workshop participants agreed that a collaborative approach to optimise pillar design will be desirable and may be possible in future. The Mandela Mining Precinct, under the auspices of the Minerals Council of South Africa, can play a leading role in this regard. Sufficient funding is currently lacking and there should be an attempt to identify possible funding sources and current systems and facilities that can be used. During the workshop, the following questions were raised and it is discussed in further detail in the section on the "Way Forward":

- Are the timelines realistic?
- Is the topic still going to be useful if the timeline is pushed out?
- Is the funding allocated to the project sufficient?
- There is a significant amount of testing that needs to be done. Is there surface and underground "laboratories" where the testing can be done?
- Considering how specific the criterium is, is a single test site sufficient?

Based on the information obtained from the workshop, it was decided to send a questionnaire to industry to obtain the required technical detail. It proved to be difficult to obtain this information from all participants in a workshop with a finite time duration. At the time of compiling this report, a total of ten questionnaires were returned. Although this may appear to be a small sample size, it covers all major mining groups using bord and pillar mining operations. Responses were also received from all the major commodities which are exploited using these mining methods, namely, manganese, platinum, and chrome.

A summary of the responses are given in Table 1 below.

Table 1: Summary of information obtained from the questionnaires.

NEED TO OPTIMIZE DESIGN	Yes	Yes	Yes	Done on a continuous basis	Yes	Possibly	Yes	Yes	Probably not, but possible	Yes
NUMERICAL MODELLING	No	Yes (FLA3D, 3DEC, MAP3D)	No	Yes (MINSIM, TEXAN, FLA3D)	Yes (TEXAN)	Yes (MAP3D, FLAC2D)	Yes (TEXAN, FLAC3D)	Yes (MAP3D, FLAC2D)	Yes (MAP3D, TEXAN)	Yes (UDEC, BESOL)
PILLAR COLLAPSE	No	No	No	No, only localized instability	Old sections	No	No	No	No	Yes
PILLAR SCALING	Few areas	No	No	Yes, typical for the depth	Old sections	Rarely	No	No	No	Yes caused by weak layers
BORD WIDTHS	8m	10m	7m	7m	8m	10m	6m-10m	10m x 10m or 10m x6m	Up to 14m	6m, 8m in small area
MINING HEIGHT	3.6m-5m	2.2m	2.4m	1.8m-2.2m	1.9m-3.5m	2.4m	1.7m-2.5m	2m	1.2m-2m	1.7m-2m
PILLAR DIMENSIONS	From 6m x 6m to 8m x8m	8m x 8m	7.5m x 7m (350m depths)	7m x7m to 8.5m x 8.5	7m x 7m	4.5m x 4.5 to 9m x 9m	8m x 8m at 410m depth and s/w=2.5	Varying	Site specific	6m x 6m to 8m x 9m changed to 8m x 8m with modelling
FACTOR OF SAFETY	1.5	1.6 (2 for declines)	1.5	N/A	1.5 (2 for declines)	1.6	1.6-3	1.6-3	1.6 or 2 (weak layers)	1.5
OVERBURDEN DENSITY	2800 kg/	3000 kg/	3200 kg/	2850 kg/	2900 kg/	2850 kg/	3000 kg/	3000 kg/	3800 kg/	3000 kg/
WEAK LAYER IN PILLARS	No	No	See comment 1	No hourglass figure	Some areas	No	No	No	Some areas	One area
UCS OF REEF	330 MPa	91 MPa	110 MPa	112 MPa	91 MPa	150 MPa	70-90 MPa	100-150 MPa	80-120 MPa	78.8 MPa
K-PARAMETER	133 MPa	48 MPa	36 MPa (0.33 *UCS)	N/A	Third of the UCS	64 MPa	57.5	53 MPa MG1/2 33.9 MPa LG6/6A	67 MPa PlatMine formula	2/3*UCS
ALTERNATE DESIGN METHODOLOGY	No	No	No	Numerical modeling	No	No	No	No	No	Numerical modeling
TYPE OF FORMULA	Hardley and Grant	Hardley and Grant	Hardley and Grant	N/A	Hardley and Grant, but considering platinum mine	Hardley and Grant	Hardley and Grant	Hardley and Grant	PlatMine UG2	Hardley and Grant
EMPIRICAL FORMULA USED?	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
RANGE OF DEPTHS	180m-800m	60m-330m	35m-400m	850m-1350m	Surface-800m	50m-650m	30m-500m	30m-600m	30m-800m	30m-350m
MULTI-REEF MINING?	No	No	No	No	No	No	No	Yes	No	No
REEF TYPE	Manganese	Chromitite	LG6/LG6A	Merensky	UG2	UG2	LG6/LG6A	MG1/2,LG6/ LG6A	UG2	LG6/LG6A
COMMODITY MINED	Manganese	Chrome	Chrome	Platinum	Platinum	Platinum	Chrome	Chrome	Chrome	Chrome

From the workshop and questionnaire responses, a number of clear trends emerged. These findings are interesting as it includes different commodity types with different rock strengths.

- There is unanimous agreement that there is a need to optimise the pillar designs.

- Most of the operations currently still use the empirical Hedley and Grant pillar strength formula for pillar design. One operation reported the use of the "Stacey and Page" formula (Stacey and Page, 1986), but this is essentially the Hedley and Grant formula where the "design rock mass strength" (DRMS) is used for the K-value.

- Regarding the K-values used in the Hedley and Grant formula, it ranges from 33.9 MPa to 133 MPa. This value has been optimised in some operations using numerical modelling and back analysis, but the old rule of a third of the UCS is still applied in many cases. One operation uses the "Stacey and Page" formula, so it is presumed that the K-value is derived from the DRMS at this operation.

- The bord widths range from 6 m to 10 m at the operations.

- Mining heights are typically in the range of 1.8 m to 2.4 m. In a few cases, heights exceeding 3.5 m and even up to 5 m were reported.

- Typical factors of safety are 1.5 to 1.6. Higher values are used where important excavations, such as declines, need to be protected.

- Although a function of depth, the most common pillar dimensions at the current depths are 7 m x 7 m or 8 m x 8 m.

- Regarding depths, the planned depths range from surface to 600 m. One operation plans to use the layouts up to 800 m depth. As an outlier, one operation uses these layouts at depths between 850 m – 1350 m. A different design philosophy, and not Hedley and Grant, is used at this operation.

- Numerical modelling is used on occasion to analyse pillar behaviour and popular codes are TEXAN and FLAC.

- Although limited scaling of pillars have been observed in isolated cases, only one industry participant reported observations of pillar collapses (excluding the three well know large mine collapses caused by weak layers). The absence of failed pillars is problematic for future studies as the statistical methods used by for example Salamon and Munro to derive their famous coal pillar equation cannot be repeated for the current bord and pillar operations. Alternative methods to determine pillar strength will have to be developed.

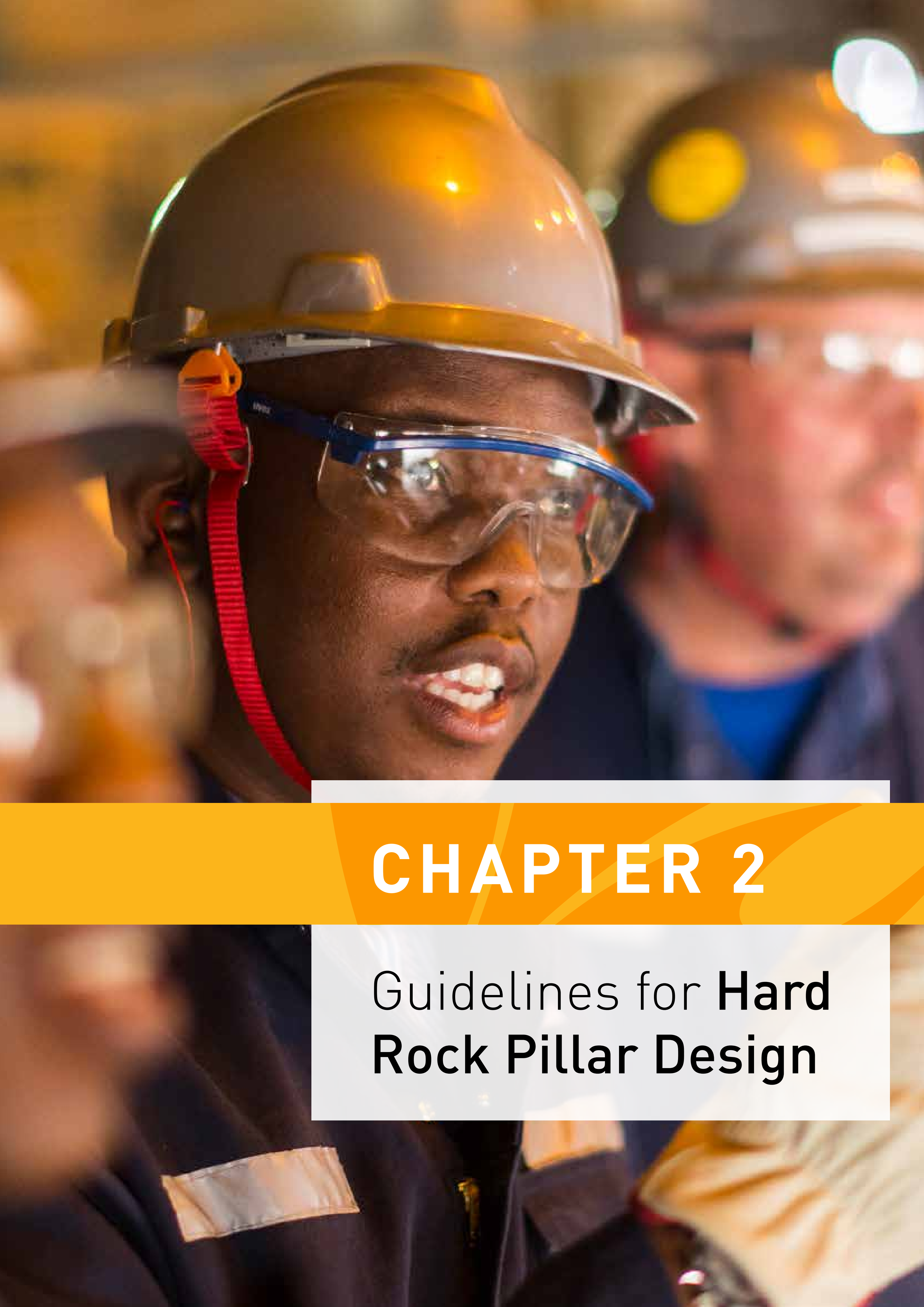
1.6 CONCLUSIONS

The objective of this project was a review of the rock engineering principles that may affect the planned mechanisation of South African tabular mines. The key aspects considered were larger pillar sizes and lower extraction ratios at increasing depths for bord and pillar layouts, the principles of support design for mechanised mining, a possible change in seismic behaviour in deep mines if the operations are mechanised and the use of numerical models for pillar design. Based on the information collected, it was agreed that a lack of knowledge on hard rock pillar strength was the most pressing issue to address in this project and in future research. Improved pillar strength formulae are required to optimise bord and pillar layouts at greater depths. The workshop with industry and a questionnaire therefore only focussed on this topic, as well as the guidelines and proposed future research presented in this document. Some of the information from a previous milestone report is included in this document to make it a standalone report.

No analytical solution to describe the effect of depth on the extraction ratio for a bord and pillar layout is readily available in literature. Such a solution was derived for this project and the results indicate that the maximum depth at which an acceptable extraction ratio can be achieved is strongly dependent on the K-value used in the pillar strength formula. The uncertainty regarding appropriate formulae and K-values makes an estimation of viable extraction ratios at depth exceedingly difficult. This is a significant hurdle in terms of future mechanisation of the South African mines. Successful mechanisation may also be impacted if there is a very weak contact (parting plane or clay layer) present in the pillars. The pillar strength may be compromised to such an extent that normal bord and pillar layouts with an acceptable extraction ratio may not be feasible. Further research needs to be conducted to determine the pillar strength in areas where weak layers are present.

The important modelling tools available for simulating pillar stress and strength can be grouped into boundary element and finite difference approaches. Boundary element models are useful when computing the closure and stress distribution across large-scale layouts. Finite difference codes are useful to simulate the failure mechanisms in a single pillar. This report illustrates the important applications of both tools. The TEXAN code with the limit equilibrium model can accurately simulate pillar stress and pillar failure in a large-scale model containing many pillars. In contrast, the FLAC code is useful to simulate pillar failure mechanisms, such as the effect of a weak layer on pillar strength.

As part of the project, it was necessary to obtain up-to-date information from the industry on the methodologies used for pillar design. A workshop was held on 5 October 2020 with rock engineering practitioners and a follow-up questionnaire was sent to industry to determine the latest industry best practice in terms of pillar design. From the data collected, it is striking that most of the operations still use the empirical Hedley and Grant pillar strength formula for layout designs. This wide adoption continues regardless of the knowledge that the formula has limitations and that there is a need to optimise the pillar designs. Numerical modelling is currently only used to a limited extent to optimise the designs. The K-values used in the Hedley and Grant formula ranges from 33.9 MPa to 133 MPa. This value has been optimised in some operations using numerical modelling and back analysis, but the old rule of a third of the UCS is still applied in many cases. The bord widths range from 6 m to 10 m and mining heights are typically in the range of 1.8 m to 2.4 m. Typical factors of safety are 1.5 to 1.6. Higher values are used when important excavations, such as declines, need to be protected. Although a function of depth, the most common pillar dimensions are 7 m x 7 m or 8 m x 8 m and the planned depths for these layouts range from surface to 600 m. Of significant interest is a particular operation that uses pillar layouts at depths of 850 m to 1350 m to extract the Merensky Reef. Although pillar scaling is reported for the 7 m to 8.5 m wide pillars at this operation, large-scale instabilities have not been reported yet. This is possibly an indication that the Hedley and Grant approach may be too conservative for some mining areas.



CHAPTER 2

Guidelines for **Hard Rock Pillar Design**

An important aspect of this project was the identification of knowledge gaps in terms of hard rock pillar strength. It is therefore important to note that the guidelines given below are only provisional. A proposal for additional research into pillar strength is given in the next section, and until additional knowledge is generated by this research, operations need to do their own due diligence to ensure the pillar systems used at the various mines are stable.

1. From the questionnaires completed by industry, it is clear that the Hedley and Grant power-law strength formula is currently still best practice in South Africa in terms of pillar design.

2. The empirical Hedley and Grant formula was derived for Canadian uranium pillars and it is used in South Africa for different reef types and outside of the parameter limits for which it was developed. It should therefore always be used with caution.

3. A pillar monitoring programme should be initiated in all mines where the formula is used. Pillar stability should be monitored on an ongoing basis.

4. In many cases, where the pillar strength is not reduced by weak layers or other geological structures, the classical assumption of seems to be conservative and many mines have increased this K-value using numerical modelling and back analysis. If this is implemented, a trail area should be mined using the modified equation with appropriate instrumentation installed. Numerical modelling back analysis should also be conducted.

5. Monitoring of pillar stability should include visual observations, closure monitoring (virtually no ongoing closure should be recorded in a stable bord and pillar back area) and stress measurements (to verify the modelling results and overburden density assumptions). The numerical modelling should be done with a code that can solve a large number of pillars of irregular size and be able to accurately compute the APS values.

6. To determine the APS on pillars, especially for back analysis, tributary area theory (TAT) is not considered accurate enough owing to the presence of large unmined areas, the finite dimension of the mining area and the effect of abutments. Numerical modelling with a code, such as TEXAN or MAP3D, should be done.

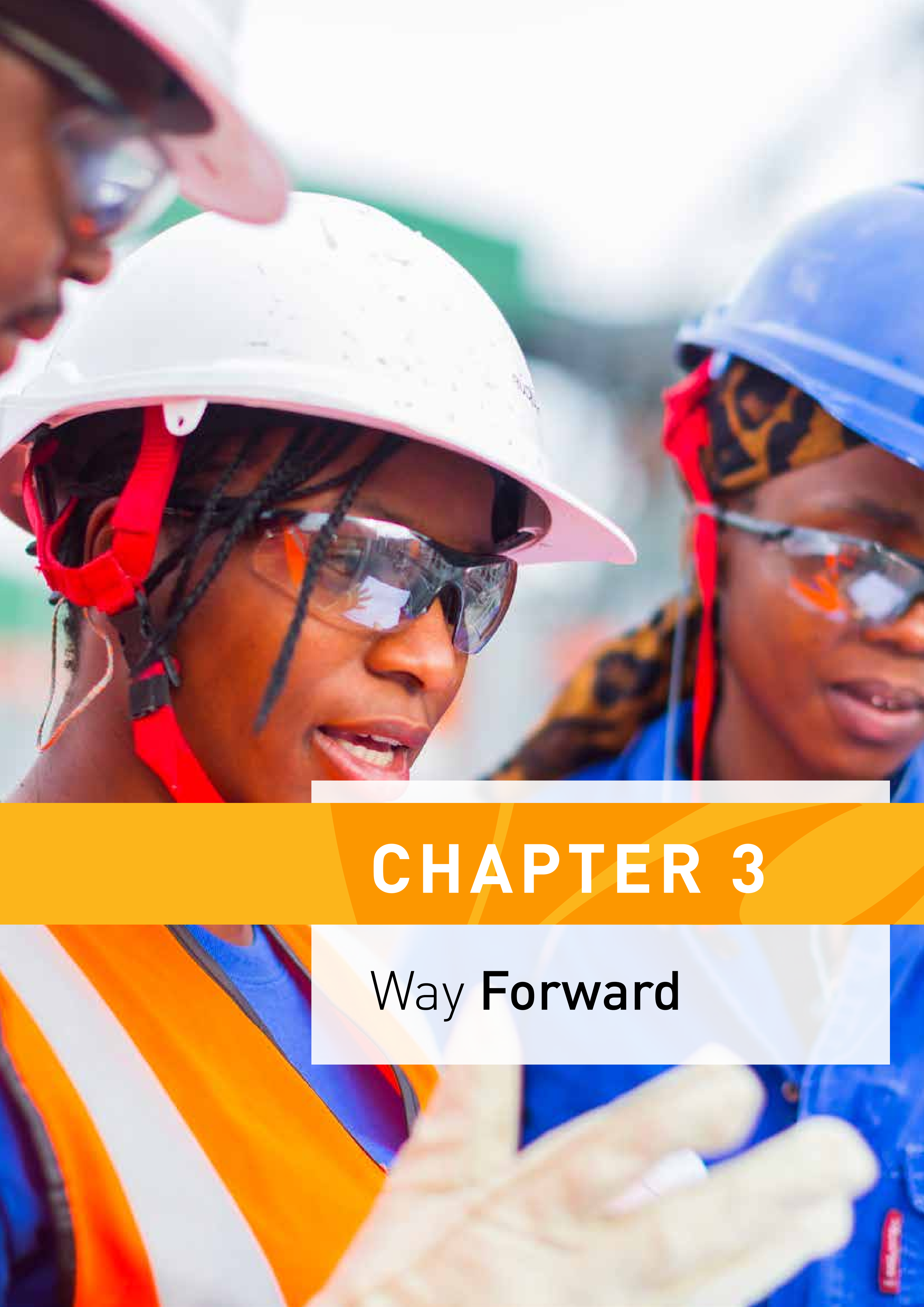
7. For pillar design or back analysis, APS values calculated by displacement discontinuity numerical modelling codes will be dependent on element sizes. Studies indicated that more accurate values are obtained as the element size tends to zero. Results obtained from finite difference or finite element codes may also be affected by element sizes. This effect should be evaluated in the codes used to minimise the errors.

8. If weak layers are present in the pillars, use of the Hedley and Grant formula may lead to mine-wide collapses. An alternative design methodology, or even an alternative mining method, will have to be considered for these ground conditions. At the very least, a system of large barrier pillars is required.

9. If the PlatMine formulae are considered for use, it should be recognised that, because of the exponents used for the width and height parameters, the formulae predicts an ever increasing pillar strength for an increase in volume (if the w:h ratio remains constant). This is considered unrealistic. These formulae also predict pillar strengths significantly larger than the Hedley and Grant formula. This again emphasises the need for monitoring and mining of a trial section as described in bullet number 4.

10. Numerical techniques to design pillar systems should be used with caution. These techniques also rely on many assumptions. Many of the approaches described in literature retains the flavour of a "semi-empirical" approach. As an example, some authors expressed some concern about the applicability of the Hoek and Brown failure criteria used in many of the numerical modelling codes.

11. As there is still significant uncertainty regarding pillar strengths, designs should preferably be done using a combination of empirical approaches and numerical modelling. Numerical modelling is valuable to investigate the possible mechanisms of failure, e.g. in the cases where weak layers are present, and will provide insight whether the empirical strength estimates are appropriate.



CHAPTER 3

Way **Forward**

Based on the findings in this report, it is clear that most of the industry still uses the same empirical design approach that was introduced in the 1970s. Substantial benefits may be derived if enhanced pillar design methodologies can be developed. Pillar strength is a very difficult rock engineering problem and it will require a collaborative effort to tackle this aspect. It is therefore recommended that a future, industry-wide project under the auspices of the Mandela Mining Precinct, needs to be executed to develop improved pillar design methodologies. Developing new design methodologies will require a large volume of underground data from operations exploiting different reef types. The only feasible method to obtain this data will be if the mining industry gets actively involved in this initiative. Owing to limited research budgets and manpower, this will have to involve mining personnel, and not only researchers, collecting the data. Encouraging was that the workshop participants supported the concept of future collaboration in industry to address this problem. The collaborative research project needs to cover the following aspects:

3.1 REEF TYPES TO BE INVESTIGATED

The pillar strength and behaviour will most likely be different for the different reef types and projects therefore need to be scoped for the following reefs. It is envisaged that at least five different formulae, or at least a suitable empirical formula calibrated for the five different reefs, to be developed. These orebodies are:

1. Merensky Reef
2. UG2 Reef
3. MG chrome ore
4. LG6/LG6A chrome ore
5. Manganese ore

Figure 18 is a graphical representation of three different pillars to illustrate that a difference in pillar behaviour under load can be expected. It is striking that the same Hedley and Grant empirical formula was used to design the sizes of all three pillars in the photographs.

a) LG6/LG6A pillar with internal waste

b) Manganese pillar with prominent joints

c) Pillar in the Great Dyke in Zimbabwe

Figure 18. Different pillar types. A similar power-law strength formula is probably not applicable to all these pillars.

3.2 LABORATORY TESTING WORK

A key aspect to be addressed is the form of the pillar strength formula. It needs to be addressed if a power-law formula is required or if the simpler linear from of the equation is more suitable (e.g as proposed in Figure 8 by some authors). This work can realistically only be determined using laboratory testing. Historically, some laboratory testing on different size cube specimens were conducted. This work needs to be repeated for the reef types mentioned above. The work required will be the following steps:

1. Conduct a detailed literature survey on the historic testing of laboratory specimens to determine the strength behaviour as the specimen sizes increased and when the width to height of the specimens were varied. This historical information will guide the planned testing programme and sample preparation.

2. Examine the laboratory testing facilities available in South Africa to determine the capabilities still remaining in the country. It needs to be determined what the maximum size of specimens is that can still be tested.

3. The next step will be sample collection and preparation. Logistically, this may also be a difficult step depending on the sample sizes required.

4. Conduct the laboratory testing as required. As this will be a very expensive exercise, careful planning of this step is required to ensure that the required data is collected.

3.3 UNDERGROUND BACK ANALYSIS OF SMALL PILLARS

The approach followed by Salamon and Munro was the collection of a large database of failed and intact pillars and, by adopting the power-law formula and using statistical methods, the parameters of the formula was calibrated. From the industry workshop and questionnaire completed, it is clear that there are almost no cases of pillar collapses recorded by the current rock engineering practitioners. A similar statistical approach can therefore not be followed. As an alternative, the following is proposed:

Careful numerical modelling of the underground layouts and the pillar APS needs to be conducted using a code such as TEXAN (see Figure 19). The actual pillar shapes need to be used in the model and the abutments and large unmined areas included.
In most mines there are pillar cut smaller than the design specifications. If these pillars are still intact, the simulated APS value will be at least the minimum strength of the pillar. The small pillars need to be simulated and inspected underground to record their condition.
A database of the minimum pillar strength plus pillar dimensions can then be compiled. This is an alternative database to estimate the pillar strength. Although it may be a conservative strength, this can be used as a first calibration of the pillar formula developed from the laboratory testing.

This approach has already been used in industry to increase the K-value in the Hedley and Grant formula. This work needs to be repeated for the five reef types mentioned above and be conducted on a much larger scale. Ideally small pillars with different mining heights should be analysed, although it will be difficult owing to the small variation in mining height used in most mining operations.

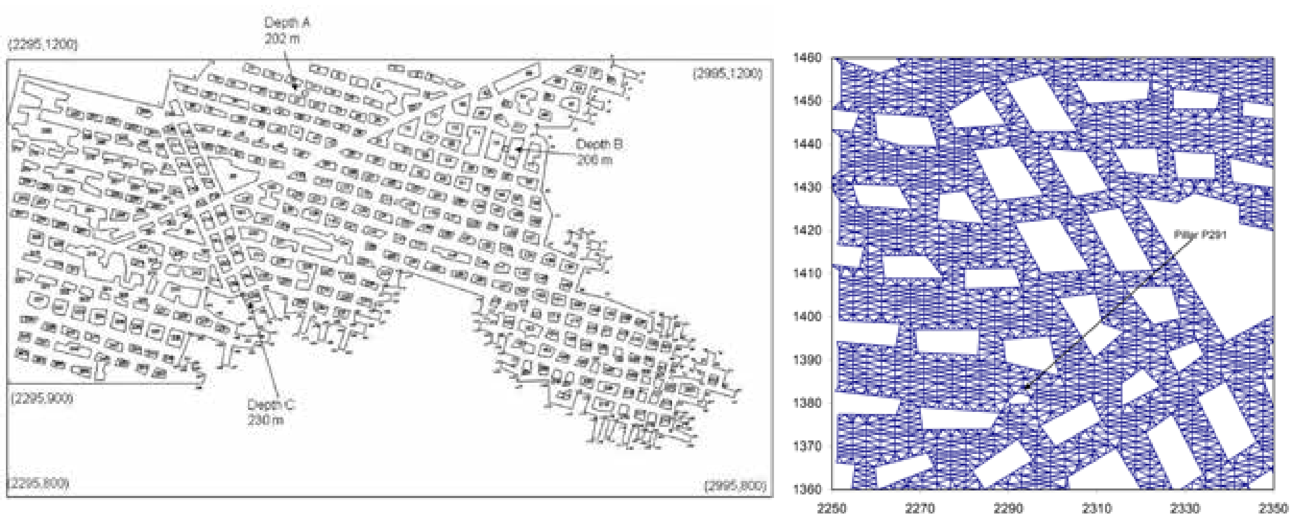


Figure 19. Example of careful modelling of actual pillar shapes and the layout to determine the APS on a small pillar.

3.4 UNDERGROUND EXPERIMENTAL WORK BY INDUCING PILLAR FAILURE

It is envisioned that a preliminary pillar strength formula can be developed from the work outlined above. A further step using underground experimentation is required as a verification of any new pillar formula. A number of mines have already embarked on developing experimental sites in older parts of their mines where a specific pillar is instrumented. The adjacent pillars are typically gradually mined to increase the spans surrounding the central instrumented pillar and increase the load acting on this pillar. In theory, the peak strength of this pillar can then be determined from stress change instruments. This strength can be compared to the strength predicted by the new pillar stress formula for the particular reef type. These experiments are very expensive as pillars need to be mined in a controlled fashion under safe conditions. The duration of these experiments can easily exceed several months. The instrumentation required and the routine collection of the data is also expensive.

3.5 INVESTIGATING THE EFFECT OF WEAK LAYERS ON PILLAR STRENGTH

As described above, the presence of weak layers can reduce the pillar strength substantially and this led to three major mine collapses in recent times in Southern Africa. It is recommended that the work outlined above be conducted for "normal" pillars where these weak layers are not present. Research work then needs to be conducted to determine appropriate modification factors to apply to pillar strength where the weak layers are present. It also needs to be determined if the expected increase in strength for an increase in pillar width apply in these cases. The work needs to consist of two components namely:

1. Laboratory testing work where such a slippery interface is simulated in the specimen.

2. Numerical modelling to simulate the effect of weak layers.

3.6 FORMAL SCOPING AND COSTING OF THE PROJECT

As the project outlined above is very detailed and costly, it is recommended that a small project be conducted initially (typically 3 months in duration) to scope the larger project. Aspects, such as determining the type of testing facilities still available and conducting the literature study of the historic testing of large scale laboratory specimens, needs to be conducted in this preliminary project. The human resource requirements can also be determined. There is scope that post graduate students at a university be used for this type of research and the importance of the topic may warrant the establishment of a SAMERDI Research Centre at a University.

Some of the questions posed at the workshop with industry were the following (the questions are given in italics and the preliminary answers are given adjacent to these questions):

- **Are the timelines realistic?** It is envisaged that such a project will be able to produce good results in a period of 3 to 5 years if sufficient resources are allocated to do the work. The 5 year duration is preferable as PhD students typically require more than 3 years for in-depth studies.

- **Is the topic still going to be useful if the timeline is pushed out?** This can be determined in the small scoping project done upfront. It is expected that the life of mine of most new operations far exceed 5 years and as they will be deeper at the end of the project, the work will be very valuable to these operations.

- **Is the funding allocated to the project sufficient?** The funding required will have to be determined in the scoping project. At the workshop some indication of funding required was given. One of the mining groups conducted a pillar reduction experiment with extensive monitoring and this required funding of R20 million over the last three years. If this is repeated for five different reef types, it will be a total cost of R100 million. If R10 million is added for the complex laboratory testing and sample preparation, R10 million for a University Research Centre to train and involve post graduate students and R30 million for the funding of the researchers over a five year period, this project will typically cost R150 million. This should be compared to the potential benefit gained by industry. The mine that conducted the experimental pillar reduction work reported that they estimated that this will add an additional $1.3 billion in revenue over the 25 year life of mine.

- **There is a significant amount of testing that needs to be done. Is there surface and underground "laboratories" where the testing can be done?** This needs to be determined in the scoping project. The laboratory at the University of the Witwatersrand is still available and there are privately run laboratories in Pretoria and in Johannesburg. The Civil Engineering Departments at the universities, such as the University of Pretoria, typically also has testing facilities available.

- **Considering how specific the criterium is, is a single test site sufficient?** This is described in detail above. A single test site will not be sufficient and the work needs to be repeated for the different reef types.

CHAPTER 4

Recommendations

As a next step, a scoping project under the auspices of the Mandela Mining Precinct needs to be conducted to determine the requirements for the proposed large pillar strength project in the South African mining industry. As the Hedley and Grant formula may be conservative in many cases, the benefits of such a project may be significant for South Africa.

In conclusions, it is clear that additional research into this topic is required and significant benefits from this may be derived by industry. A project with a longer term duration will also be beneficial to rebuild research capacity in South Africa. In a recent article, Stacey (2019) wrote: "How is it possible that the mining industry would regard the research as too costly? And thus, how can rock engineering research capacity in South Africa have been allowed to dissipate completely from the research powerhouse that it once was?" A similar sentiment was shared by Van der Merwe (2006) who used the Coalbrook disaster as an example of lessons not learnt from disasters and deplored the dismal state of rock engineering research in South Africa. He stated: *"Is it conceivable that the most important lesson from Coalbrook, namely that in order to be effective at all, knowledge has to be generated before it is needed, was not learnt?"*

REFERENCES

Bieniawski, Z.T., 1992. A method revisited: coal pillar strength formula based on field investigations. In *Proceedings of the workshop on coal pillar mechanics and design. Pittsburgh, PA: US Department of the Interior, Bureau of Mines, IC* (Vol. 9315, pp. 158-165).

Day, A.P. and Godden, S.J., 2000. The design of panel pillars on Lonmin Platinum's Mines. In *Proceedings SANIRE 2000 Symposium, Rustenburg.*

Esterhuizen, G.S, 2006, An evaluation of the strength of slender pillars. *SME Annual Meeting, St. Louis, Missouri.*

Hedley, D G F and Grant, F, 1972. Stope-and-pillar design for Elliot Lake Uranium Mines. *Bull. Can. Inst. Min. Metal.*, 65, pp 37-44.

Hoek, E., Kaiser, P.K. and Bawden, W.F., 1995. Support of underground excavations in hard rock: Rotterdam. *AA Balkema*, pp.84-97.

Hustrulid, W.A., 1976. A review of coal pillar strength formulas. *Rock Mechanics*, 8(2), pp.115-145.

Kumar, A, and Singh, R., 2018. Numerical modelling based investigation of coal pillar stability for room and pillar development at 900m depth of cover, *37th International Conference on Ground Control in Mining, Morgantown, West Virginia.*

Lousteau, R.R. (2016) Geotechnical design for the Booysendal Central Block, Northam Booysendal Division, Middindi Consulting (Pty) Ltd, Report No MDI 2015 – 0078.002 , 21 April 2016.

Malan, D.F. and Napier, J.A.L. 2011. The design of stable pillars in the Bushveld mines: A problem solved? *J. S. Afr. Inst. Min. Metall.*, vol. 111, pp 821-836.

Martin, C D and Maybee, W G, 2000. The strength of hard rock pillars, *Int. J. Rock Mech Min. Sci*, vol. 37, pp 1239 – 1246.

Mostyn, G and Douglas, K, 2000, Strength of intact rock and rock masses. *GeoEng 2000, Melbourne*

Napier, J.A.L. and Malan D.F. 2018. Simulation of tabular mine face advance rates using a simplified fracture zone model. *International Journal of Rock Mechanics and Mining Sciences* 109:105–114.

Napier, J.A.L. and Malan D.F. 2020. A limit equilibrium model of tabular mine pillar failure. *Rock Mechanics and Rock Engineering. https://doi.org/10.1007/s00603-020-02265-2*

Priest, S D and Brown, E T, 1983. Probabilistic stability analysis of variable rock slopes. *Trans Inst. Min. Metall.* 92, A1-A12.

Ryder, J. A., and Jager, A. J. (2002). Rock Mechanics for Tabular Hard Rock Mines. *Johannesburg. SIMRAC.*

Ryder, J.A. and Jager, A.J., 2002. A textbook on rock mechanics for tabular hard rock mines. *SIMRAC, Johannesburg*

Salamon, M D G and Munro, A H, 1967. A study of the strength of coal pillars, *J.S. Afr. Inst. Min. Metall.*, vol. 68, pp 56 – 67.

Spencer, D, 1999. A case study of a pillar system failure at shallow depth in a chrome mine, In: Hagan, T.O. (ed.), *Proc. SARES99, 2nd Southern African Rock Engineering Symposium*, pp 53-59.

Spencer, D.A. (2009) Rock engineering designs for the UG2 North and South Sections of the Booysensdal project. Spencer Rock Mechanics Consultancy, June 2009.

Spottiswoode, S.M. and Milev, A. M. (2002). A methodology and computer program for applying improved, inelastic ERR for the design of mine layouts on planar reefs. *SIMRAC GAP 722 Final Report. Johannesburg. SIMRAC.*

Stacey, T.R. and Page, C.H. 1986. Practical handbook for underground rock mechanics. *Trans Tech Publications, Germany.*

Watson, B.P., Ryder, J.A., Kataka M.O., Kuijpers, J.S. and Leteane F.P. 2008. Merensky pillar strength formulae based on back-analysis of pillar failures at Impala Platinum, *J. S. Afr. Inst. Min. Metall.*, vol. 108, pp 449-461.

Watson, B.P., Theron, W., Fernandes, N. and Roberts D.P. (2020) Pillar Strength Formulae for the UG2 platinum reef based on back analysis of pillar failures, Paper prepared for submission. *J. S. Afr. Inst. Min. Metall.*