Thesis Summary

THE DESIGN AND BEHAVIOUR OF CRUSH PILLARS ON THE MERENSKY REEF

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ABSTRACT

Crush pillars are extensively used in the platinum mines of South Africa as part of the stope support in intermediate depth tabular mining stopes. Crush pillar design should ensure that the pillars crush when formed at the mining face. This behaviour is typically achieved when the pillars have a width to height ratio of approximately 2:1. Once crushed, the residual stress state of the pillars provide a local support function. However, in most cases effective pillar crushing is not achieved, resulting in pillar seismicity. As the area produces approximately 70% of the world’s platinum group metals, it is critical that layouts and pillar design are optimised to ensure safety and sustainable production. The objective of the research was to determine the parameters which influence crush pillar behaviour. A limit equilibrium constitutive model was proposed to investigate the behaviour of the pillars. The model, implemented in a displacement discontinuity boundary element code provided insights into the stress evolution of a crush pillar. The results indicated that the stress on the pillar depends on its position relative to the mining face, the effect of over-sized pillars, the impact of geological structures, layout, rock mass parameters and mining depth.

An underground mining trial was conducted at Lonmin Platinum to quantify the behaviour of crush pillars. This was the most comprehensive monitoring of these pillars ever conducted in the platinum industry. The observed behaviour of the pillars agreed well with the findings of the measurements and the pillar fracturing profiles obtained at various stages of the pillar forming cycle. A sequence and mode of pillar failure could be identified. The results indicated that a pillar reaches a residual stress state when separated from the mining face. The pillar also experiences a secondary reduction in stress when new pillars are formed. However, at some point, the pillars experienced no further reduction in stress while the pillars continue to deform. Ongoing convergence was also recorded after all mining was stopped.

A numerical model was used to back analyse the behaviour of the underground trial site which consisted of a mined area of approximately 22 000 m² containing 55 crush pillars. To date, no numerical modelling of a mine-wide tabular layout, which explicitly included a large number of crush pillars, had been reported. This work is therefore considered a major novel contribution to this field of research. Both the observed and measured behaviour of the crush pillars in the trial site could be replicated by the model. The findings validated the use of the limit equilibrium model implemented in a displacement discontinuity boundary element code to simulate the behaviour of crush pillars on a large scale.

This work has the potential to significantly influence the mining industry on the application of crush pillars. It could therefore increase safety and assist production in the platinum mines of South Africa.
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1. INTRODUCTION

Mining practices are aimed at maximising the extraction of a particular orebody without compromising safety. Crush pillar mining appears to be a method unique to South African hard rock mines. These pillar systems are used in shallow and intermediate depth platinum stopes. It allows for a higher extraction than what can typically be achieved with a conventional elastic non-yield pillar system. The pillar system must, however, be used in conjunction with a regional pillar system. Crush pillar dimensions are generally selected to give a width to height ratio (w:h) of approximately 2 (Ryder and Jager, 2002). This w:h ratio should ensure that the pillars fail while being cut. Once the pillar has failed in a stable manner, the residual strength of the pillar contributes to the support requirements by carrying the deadweight load to the height of the uppermost parting on which separation is expected to occur (can be as much as 45 m above the reef). The pillars therefore prevent the occurrence of large scale collapses (backbreaks). Closely spaced support elements are typically used between rows of crush pillars to provide additional in-panel support.

Figure 1.1: Photograph of a crush pillar from a trial site on Lonmin.
A typical mining configuration for an in-stope pillar layout in a hard rock narrow tabular mine (stopping width = 0.9 – 2 m) is shown in Figure 1.2. The pillars are separated in the direction of mining by a holing, 2 – 4 m wide. Inter-pillar spans vary between 20 – 35 m. An off-reef haulage links to the reef horizon via a cross-cut and travelling way.

Figure 1.2: Example of an in-stope pillar layout (plan view) for narrow tabular reef mining. The diagram indicates a down-dip split-panel layout as used in the trial mining section on Lonmin (average dip of approximately 8°).
Figure 1.3 illustrates the stress-strain relationship of a typical pillar. The initial straight line portion of the curve to the yield point reflects the elastic response of the pillar. The yield point indicates the onset of inelastic behaviour whereafter the pillar exhibits strain hardening until it reaches its peak strength. Load shedding follows until the pillar reaches its residual strength. Crush pillars are designed to function in this residual part of the pillar stress-strain curve.

![Complete stress-strain curve of a pillar (after Ryder and Jager, 2002).](image)

Although crush pillars has extensively been used on the Merensky Reef horizon (main platinum reef) since the late 1970’s, little is known about their behaviour and no design methodology exists. The “design” is still predominantly limited to specifying a width to height ratio of approximately 2:1 and the assumption that pillars should be crushing close to the face whilst being cut.

Several factors affect the behaviour of the crush pillars and in many cases satisfactory pillar crushing is not achieved. Also, the implementation of a crush pillar system on many mines is problematic owing to the difficulty of controlling pillar sizes with poor drilling and blasting practices. This results in a seismic risk in many of the mines using crush pillars. If pillar crushing is not initiated whilst the pillar is being formed, as the mining face advances and the pillars move to the back area of a stope, some pillars may burst while very-large pillars may punch into the footwall (e.g. Figures 1.4 and 1.5).
Crushed rock in dynamic shear defined by two parting planes mobilised during footwall heave.

Figure 1.4: Example of pillar foundation failure. The crushed rock is contained in the siding between the pillar (not visible on the far left) and the grout pack.

Figure 1.5: Example of pillar bursting. The scattered pillar material was ejected into the siding. The white lines indicate the ejected material adjacent to the pillar (left) relative to the stoping width (right). The timber elongates indicate dynamic loading as a result of the event.
The objective of this study is to address many of the shortcomings regarding the understanding of crush pillar behaviour. It is necessary to understand how the pillars behave and to investigate the parameters which will affect pillar crushing. On most operations where crush pillars are used, pillar crushing is not achieved. Crush pillars are implemented at relatively shallow depths, the pillar dimensions have remained essentially unchanged over many years and the impact of regional pillars and geological losses contributing to the regional behaviour of the rock mass are overlooked. Furthermore, in many cases the pillar system is the source of seismicity.

The study considered the risk of pillar instability and focussed on establishing design criteria with regard to pillar size and stress, namely:

- The effect of pillar width,
- The impact of mining depth on pillar crushing,
- Determining the peak crush pillar strength,
- Determining the residual crush pillar strength,
- Determining if and how convergence is related to pillar crushing,
- The effect of oversized pillars,
- The impact of geological and mining losses on pillar crushing.

The outcome of the research provided an improved understanding of the behaviour of crush pillars on the Merensky Reef and the factors influencing the pillar behaviour. The pillar behaviour of other reef types is considered beyond the scope of this study.
2. SIMULATION OF CRUSH PILLAR BEHAVIOUR

A major challenge in the design of pillar layouts is to integrate an appropriate representation of the pillar failure behaviour with the overall analysis of the tabular mining stress distribution. As crush pillars consist of crushed reef material, the detailed inelastic analysis of reef crushing behaviour is most appropriately conducted by means of non-linear finite element or finite difference models. However, for tabular excavation layouts, stress interactions in three dimensions can be efficiently represented using a boundary element model based on displacement discontinuity elements.

Simplification to the problem is possible if the fractured reef material is known to be concentrated within or close to the mining horizon and does not extend appreciably into the hangingwall or footwall regions. This will often be the case when considering the design of crush pillars in shallow mining layouts. Various strategies have been suggested to represent non-linear seam behaviour using limit equilibrium methods (e.g. Brummer, 1987) or the so-called “enhanced” displacement discontinuity method (Yacoub and Curran, 1999) in which reef-parallel strain components are included in the set of unknown variables that are evaluated on the reef horizon. In this study an analytic limit equilibrium model, similar to coal seam deformation models introduced by Barron (1984), is employed to represent the behaviour of the reef material in pillars. Malan and Napier (2006) did a preliminary evaluation of a similar model and demonstrated that it is feasible to model pillar crushing. The original evaluation of this model is expanded on in this section. Salamon (1992) introduced a refinement of the Barron model which he called the “enhanced confined core concept” of pillar strength. The focus of the study is to illustrate the value of these limit equilibrium models when implemented in boundary element codes, such as TEXAN, to represent the behaviour of the reef material in the pillars.

2.1. An overview of the TEXAN code

In the displacement discontinuity boundary element method, mine layouts are approximated as irregular shaped planar cracks (or slits) where the ‘width’ of the crack, corresponding to the excavation height, is assumed to be negligible compared to the in-plane, lateral dimensions. The TEXAN (Tabular EXcavation ANalyser) code (Napier and Malan, 2007) utilises a general revision of this approach in which triangular or quadrilaterial element shapes are introduced in conjunction with higher order variations of the displacement discontinuity shape functions. This facilitates an accurate evaluation of detailed stress and displacement components close to excavation surfaces and allow for the assessment of tabular layouts which includes a large number of pillars.
2.2. Formulation of the limit equilibrium model

A pillar can be divided in a large number of vertical slices (Barron, 1984). The elastic stresses acting on the pillar can be analysed and compared to an appropriate failure criterion. Once a slice has failed, its residual stress is calculated. This is compared to the initial elastic stress of which the difference is transferred to the remaining unfailed slices. The successive analysis of each slice determines the depth of the failed zone, the final stress distribution, the pillar strength and for the case of failed pillars, the residual strength.

Malan and Napier (2006) illustrated the force equilibrium of a material “slice” of a fractured pillar as shown in Figure 2.1. The slice of fractured material has a mining height $H$ at a distance $x$ from the stope face. The slice is confined by reef-parallel and reef-normal stress components $\sigma_s$ and $\sigma_n$ respectively as well as by shear tractions $\tau$.

![Figure 2.1: Force equilibrium of an elementary material slice between two bounding surfaces (after Malan and Napier, 2006).](image)

It is assumed that the edge of the pillar is at $x = 0$ and that the seam-parallel stress component $\sigma_x$ is uniform over the height of the pillar and increases as $x$ increases. From Figure 2.1 it can be inferred that the equilibrium force balance acting on the slice of height $H$ and unit out of plane width requires that:

$$H\sigma_s(x + \Delta x) = H\sigma_x(x) + 2\pi\Delta x$$

(2.1)
Taking the limit $\Delta x \to 0$, equation (2.1) can be written in the form of the differential equation;

$$\frac{d\sigma_s}{dx} = \frac{2\tau}{H}$$  \hspace{1cm} (2.2)

Equation (2.2) can be solved for $\sigma_s$ if a relationship exists between $\tau$ and $\sigma_s$. This can be established by making the following assumptions:

(a) Assume that $\tau$ is related to the surface-normal stress $\sigma_n$ by a frictional slip condition of the form:

$$\tau = \mu \sigma_n,$$  \hspace{1cm} (2.3)

where $\mu$ is the friction coefficient.

(b) Assume that $\sigma_n$ is related to the average stress $\sigma_s$ by a failure relationship of the form:

$$\sigma_n = C + m\sigma_s;$$  \hspace{1cm} (2.4)

where $C$ and $m$ are specified constants.

Substituting equations (2.4) and (2.3) into equation (2.2) yields the required differential equation as follows:

$$\frac{d\sigma_s}{dx} = \frac{2\mu}{H}(C + m\sigma_s)$$  \hspace{1cm} (2.5)

Equation (2.5) can be integrated directly if it is written in the separable form;

$$\int \frac{d\sigma_s}{C + m\sigma_s} = \int \frac{2\mu}{H} dx$$  \hspace{1cm} (2.6)

Integrating equation (2.6) yields the solution to equation (2.5) in the form;

$$\ln(C + m\sigma_s) = \frac{2\mu m x}{H} + A$$  \hspace{1cm} (2.7)

where $A$ is a constant that can be determined by applying the boundary condition that the average horizontal stress $\sigma_s$ is equal to an edge constraint stress $p$ when $x = 0$. Hence, substituting $\sigma_s = p$ and $x = 0$ in equation (2.7) yields;

$$A = \ln(C + mp)$$
The average horizontal stress is then recovered from equation (2.7) as a function of the distance \( x \) from the edge of the pillar in the form:

\[
\sigma_x = (C/m + p) \exp(2\mu mx/H) - C/m
\]  

(2.8)

The seam-normal stress component \( \sigma_n \) can be obtained by substituting the expression for \( \sigma_x \) given by equation (2.8) into equation (2.4) to yield:

\[
\sigma_n = (C + mp) \exp(2\mu nx/H)
\]  

(2.9)

It is apparent from an examination of equations (2.8) and (2.9) that the solutions for \( \sigma_x \) and \( \sigma_n \) become degenerate (have zero values) if both \( C \) and \( p \) are equal to zero. In order to proceed further with the application of the simple limit equilibrium model some explicit assumptions has to be made concerning the choice of suitable values for \( C \) and \( p \). In the present approach, it is assumed that \( p = 0 \) and that the average unconfined strength of the material \( C \), at the edge of the pillar in the failed state is equal to \( C_b \). The parameter \( C_b \) is therefore equivalent to the residual unconfined strength of the crushed material (UCS_{b}). This assumption is tenable if the pillar is able to bear some vertical load at the unsupported edge \( x = 0 \). Specifically, from equation (2.9), the vertical load is \( \sigma_n = C_b \) when \( x = 0 \). If the pillar edge is, in fact, free-standing then this must be the case. However, it is apparent that the depth of the failed edge zone will be very sensitive to the exact choice of \( C_b \).

Setting \( p = 0 \) in equations (2.8) and (2.9) and defining:

\[
\alpha = 2\mu m/H
\]

yields the following expressions for the horizontal and vertical stress values in the edge region:

\[
\sigma_x = C_b (e^{\alpha x} - 1)/m
\]  

(2.10)

\[
\sigma_n = C_b e^{\alpha x}
\]  

(2.11)

The simple stress model depicted in Figure 2.1 has been extended by Salamon (1992) to allow for an explicit variation of the stress components \( \sigma_{xx} \), \( \sigma_{yy} \) and \( \sigma_{xz} \) as functions of both the edge distance coordinate \( x \) and the vertical coordinate \( z \) between the floor and roof of the pillar region. It is assumed that the material
satisfies a limit equilibrium relationship between the maximum and minimum local principal stress components $\sigma_1$ and $\sigma_3$ at each point $(x,z)$ in the failed region that is of the form;

$$\sigma_1(x,z) = m\sigma_3(x,z)$$

(2.12)

The values of the principal stress components $\sigma_1$ and $\sigma_3$ can be expressed as functions of the individual stress components $\sigma_{xx}$, $\sigma_{yy}$ and $\sigma_{xz}$ which are, in turn, defined in terms of the derivatives of an Airy stress function, $\Phi(x,z)$ that is assumed to have the form;

$$\Phi(x,s) = \exp[ax + f(z)]$$

(2.13)

Substituting these expressions into equation (2.12) allows the stress components to be solved in a closed but rather complicated form (Salamon, 1992). From the assumed relationship (2.12), it is noted that this model assumes that the cohesive strength, $C = 0$, and requires the specification of a non-zero confining stress, $p$, at the edge of the pillar. No allowance is made for roof or floor foundation failure processes and all stress components increase exponentially from the pillar edge. The added complexity of this extended limit equilibrium model is not explored in the present approach.

As pointed out by Salamon et al (2003), a Mohr-Coulomb plasticity model without strain softening behaviour is inadequate for simulating actual pillar behaviour where rapid load shedding or “bursting” may occur. To address this shortcoming, it is assumed that initial failure in the seam or reef is controlled by the additional relationship:

$$\sigma_n \leq C_0 + m_0\sigma_s$$

(2.14)

where $C_0$ and $m_0$ represent the intact strength of the pillar material. Equation (2.14) is used as well to determine implicitly the boundaries between the pillar intact core and the failed edge regions.
2.3. Analytical solution for the average pillar stress (APS) of a failed 2D pillar

Equation (2.11) predicts an exponential increase in the pillar stress away from the edge towards the centre of the pillar. In this section this model is further explored by the author to determine the stress state of a completely failed two-dimensional pillar. This model will further on be referred to as the Du Plessis model in this study.

If the pillar width is \( w \) and if the pillar is completely failed, by assuming that the stress profile is symmetric around the centre of the pillar, the average stress in the pillar (APS) is given by:

\[
APS = \frac{2 \int_{-w/2}^{w/2} \sigma_n \, dx}{w}
\]  

(2.15)

By substituting equation (2.11) into (2.15), it follows that:

\[
APS = \frac{2C_b}{\alpha} \int_{-w/2}^{w/2} e^{\alpha x} \, dx = \frac{2C_b}{\alpha \sigma w} \left[ e^{\alpha w/2} - 1 \right]
\]  

(2.16)

Substituting \( \alpha = 2\mu m / H \) into equation (2.16), the average pillar stress can be expressed as:

\[
APS = \frac{C_b H}{\mu m w} \left[ \frac{\mu m w}{e^{H}} - 1 \right]
\]  

(2.17)

The form of equation (2.17) suggests that the average stress for a completely failed crush pillar can be written compactly in the following dimensionless form;

\[
\frac{APS}{C_b} = \frac{e^\eta - 1}{\eta}
\]  

(2.18)

where the constant \( \eta = \mu m w / H \) represents a non-dimensional parameter that is proportional to the width to height ratio. In the limiting case when the width \( w \) becomes very small, \( \eta \to 0 \) and \( APS \to C_b \). This indicates that some caution should be exercised in using the simple limit equilibrium model for very slender failed pillars where \( w < H \).
2.4. Simulating a crush pillar layout

The objective of this section was to use a representative model of the behaviour of crush pillars in a typical layout. The limit equilibrium model implemented in the TEXAN displacement discontinuity boundary element (DD) code provided a numerical modelling approach that could fulfil this requirement. This provided insights into when pillars will crush, where they will crush relative to the mining face and why some pillars can potentially burst.

Various parameters were assessed to establish the factors governing crush pillar behaviour. These included rock mass parameters, mining depth (stress) and pillar geometric effects such as, pillar width, pillar height and pillar length.

The simulated layout is shown in Figure 2.2. It consists of a 30 m x 70 m stope panel with a second panel being mined in a sequential fashion adjacent to this first panel. The layout was simulated as eight mining steps with 7 crush pillars being formed during this process. For the second panel, the size of each mining step was 10 m and the sizes of the crush pillars were 4 m x 6 m. A 2 m mining height was used (w:h = 2:1). The element sizes used were 0.5 m.

The parameters used for the simulations are shown in Table 2.1. These values were chosen arbitrarily. The intent was to establish trends regarding the pillar behaviour even though the parameters selected may not fully represent the underground environment. A sensitivity analyses was conducted to determine the effect of each input parameter on the behaviour of the model and simulated pillars in the layout. The results indicated that the choice of parameters gave qualitative agreement with observed crush pillar behaviour and historic underground measurements.
Figure 2.2: Idealised crush pillar layout simulated with the TEXAN code. This is a plan view of a tabular layout.

Table 2.1: Parameters used for the crush pillar simulations.

<table>
<thead>
<tr>
<th>General Parameters</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young's modulus</td>
<td>70 GPa</td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td>0.25</td>
</tr>
<tr>
<td>Stress gradient</td>
<td>0.03 MPa/m</td>
</tr>
<tr>
<td>Depth (metres below surface)</td>
<td>600 mbs</td>
</tr>
<tr>
<td>Reef dip</td>
<td>0°</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Crush model parameters</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intact material strength $C_0$</td>
<td>5 MPa</td>
</tr>
<tr>
<td>Crushed material strength $C_b$</td>
<td>5 MPa</td>
</tr>
<tr>
<td>Intact slope $m_0$</td>
<td>5</td>
</tr>
<tr>
<td>Residual slope $m_b$</td>
<td>3</td>
</tr>
<tr>
<td>Bounding friction angle $\varphi$</td>
<td>35°</td>
</tr>
<tr>
<td>Seam height</td>
<td>2 m</td>
</tr>
<tr>
<td>Seam stiffness modulus</td>
<td>$10^6$ MPa/m</td>
</tr>
</tbody>
</table>
Various pillar sizes were simulated using the parameter set as indicated in Table 2.1. The effect of pillar width is illustrated in Figure 2.3. Pillars with a width to height ratio greater than two will most likely not crush. The wider pillars will either result in pillar crushing occurring in the back area, or not crushing at all. Figure 2.4 compares the results of three different pillar layouts, each having a different pillar width. The results are for section a – a’ through pillar D as indicated in Figure 2.2. The figure indicates the exponential stress increase towards the centre of the pillar. This is a key attribute of the limit equilibrium model. In the case of oversized pillars, this could lead to excessively high stresses in the core of wide pillars where a crushed state is not achieved. The simulated results indicated that the core of the oversized pillars were still intact. The stress profile for the w:h = 2:1 pillar is for step 5 and just before the pillar completely crushes and moves to a residual state in step 6 (compare results to Figure 2.3 indicating the stress development as a result of mining). In the back area the change in stress is lower compared to the pillar being formed at the face. In the case over-sized pillars, this may lead to violent failure in the back area of a stope if the pillar becomes more highly stressed. These pillars could potentially burst when fully loaded or punch into the footwall resulting in excessive footwall heave (Jager and Ryder, 1999).

Figure 2.3: Effect of pillar width on pillar performance (600 m below surface).
The preliminary modelling results indicated the ability of the code to simulate crush pillar behaviour. This allowed for the investigation of various parameters influencing the behaviour of crush pillars as found in typical mine layouts. The impact of the rock and pillar properties, layout parameters, mining sequence and the effect of off-reef structures in a layout could be investigated.

Design guidelines were derived from the preliminary analyses. The preliminary results based on the assumed model parameters indicated that:

- A crush pillar system should not be implemented at depths shallower than 600 m below surface (mbs).
- A maximum width to height ratio of 2:1 is required to ensure that the slender in-stope pillars crush at the face whilst they are being cut.
- If crush pillars are to be implemented at depths shallower than 600 mbs, the width of the pillars will have to be reduced (e.g. w:h = 1.5 is required at 400 mbs). This might not be practical to implement as pillar dimensions must also cater for out-of-line mining and blast effects. Furthermore the overall behaviour of the pillars could still be effected by the influence of mining losses and geological features. A minimum practical pillar width of 2 m will only crush at 600 mbs (w:h = 2:1).
- Pillar length does not appear to have a significant impact on crush pillar behaviour for pillar width to length ratios (w:l) of up to four.
The effect of unmined blocks of ground or geological losses on crush pillar behaviour, have up to now, not been considered as factors prohibiting pillar crushing or inducing pillar seismicity. For this reason it was included in the numerical analyses.

In the platinum mines, non-mineable blocks of ground are left in-situ where poor ground conditions prevail or geological features (such as potholes) are intersected. A pothole can be described as an approximately circular area where the reef slumps and pinches to such an extent that regular mining cannot be conducted. Potholes at the Lonmin site vary in size from 5 - 420 m in diameter (Figure 2.5). Most of the pothole diameters range between 20 - 100 m. The spans between adjacent potholes, in the majority of cases, are less than 100 m.

Figure 2.5: Seismic survey indicating density of potholes along the Karee block (area ≈ 6 km x 6 km). The potholes are indicated by the yellow circles.

To quantify the effect of a pothole or mining loss adjacent to a line of crush pillars, the idealised crush pillar layout (Figure 2.2) was adjusted to take these losses into consideration. The unmined block was simulated as a square block of which the dimensions [(Xm) x (Ym)] were governed by the percentage extraction locked up by the block defining mining step 1 [e.g. Pothole area (10m x 10m) ≈ 5%]. Underground potholes or mining losses may be present at various positions throughout a panel. Simulations were conducted to determine the impact of this on crush pillar behaviour.
The results highlighted the importance of taking the geological environment into consideration when implementing a crush pillar system. The pillar stress is affected by the additional stability provided by unmined blocks of ground. This causes a reduction in the pillar stress and prevents effective pillar crushing. To overcome this, smaller pillars will have to be cut, which could be impractical.

The preliminary modelling results indicated that:

- Crush pillars implemented at 600 mbs with a w:h = 2:1 will not crush if a 10% mining loss is situated adjacent to the pillar line. In this case, narrower pillars are required to ensure that pillar crushing is achieved (e.g. w:h = 1.5 is required at 600 mbs).
- Pillars situated 20 m ahead or behind an unmined block will also be affected, resulting in either partially crushed (core still solid) or intact pillars.
- Crush pillars implemented 800 m and deeper below surface are impacted to a lesser extent when in close proximity to a pothole. Large mining losses (>10%) and potholes situated closer than 10 m from the pillar line can either prevent pillar crushing or induce delayed crushing.
- Crush pillars implemented beyond 1000 mbs are not affected if a 10% mining loss (pothole) is situated 2.5 m from the line of crush pillars.

Underground sidings often significantly lag the mining face. A siding is a 1 – 2.5 m wide ledge or heading carried on the one side of an on-reef development end adjacent to the panel being mined (Figures 2.6 and 2.7). These sidings are carried a maximum of either 3 m or 6 m behind the panel face (depending on the standard applied by the respective mining company). The main function of the siding is to either modify the fracture patterns resulting from high face abutment stress or to move the crush pillars away from the travelling way to prevent failed rock falling on people. The sidings, being approximately 2 m wide, are difficult to clean (hand lashed) and support. For this reason, mining of the siding is frequently behind schedule. In some cases, sidings lag the face by 20 – 30 m and are then developed as a single mining face. A lagging siding impact the width of the pillar being formed at the mining face (Figure 2.7). Until now, the impact of a lagging siding on the pillar width has not been identified as a contributor to undesired pillar behaviour or a source of pillar seismicity in a crush pillar environment. Once the siding of an advancing panel lag behind the adjacent lagging panel face, over-sized pillars are created. The pillars will only be reduced in size to the required dimension when the siding is blasted. At this point the pillar might not be able to crush sufficiently as the pillar is already in the back area of a stope. The simulations conducted confirmed this effect (Figure 2.8). These pillars could therefore pose a seismic risk.
Figure 2.6: Section view of a typical intermediate depth mining geometry with a crush pillar layout.

Figure 2.7: The impact of a significantly large lagging siding on crush pillar behaviour (plan view).
Figure 2.8: The effect of a lagging siding on crush pillar behaviour (pillar D, Figure 2.2) at various depths (w:h = 2). Note that the pillar does not crush and shed load compared to pillar D in Figure 2.3.
2.5. Assessment of the derived analytical solution to determine the residual stress of a completely failed pillar

The analytical solution derived by the author provided an equally good fit to historic underground measured pillar behaviour (Watson, 2010) when compared to both the Salamon (Salamon, 1992) and Barron (Barron, 1984) solutions. The results are, however, very sensitive to the selection of the input parameters.

The derived analytical solution for the average pillar stress of a completely failed crush pillar was used to investigate the behaviour of these pillars. The results of the numerical modelling were compared to the analytical solution. The purpose of the analytical solutions would be to provide a first order estimate of crush pillar dimensions. The analytical solution, however, provides a two-dimensional approximation (infinitely long pillars). If very long pillars are simulated in TEXAN, the residual pillar stress of the failed pillars is in better agreement with the analytical solution (Figure 2.9).

![Graph](image)

**Figure 2.9:** Results for the analytical solution of a completely crushed pillar (equation 2.17) and numerical simulation for pillars with various width to height ratios (the same input parameters were used for both methods of analyses). The figure also compares the residual pillar stress for short versus long pillars simulated at 600 mbs.
3. UNDERGROUND TRIAL

An underground crush pillar trial site was established late 2012 on Lonmin’s K3 Shaft at a depth of approximately 782 mbs. The objective of the trial was to obtain a better understanding of the pillar system and the associated risks. The Merensky Reef, dipping at approximately 8 degrees, was mined at an average mining height of approximately 1.3 m using conventional drill and blast techniques. A split down-dip layout was used (Figure 1.2), mining from 23 to 24 level (800 mbs). Level spacings are approximately 180 m apart on dip. Split dip panels (14 m face lengths) were mined either side of a centre-raise, resulting in a maximum inter-pillar span of 31 m between lines of crush pillars. The pillars were situated in the middle of a “panel” and away from other excavations (i.e. raise) which could possibly influence the pillar behaviour. The crush pillars were planned to be 2 m wide and 4 m long (w:h of approximately 1.5).

Figure 3.1: Locality of Lonmin in relation to the Bushveld Complex.
The down-dip panels in the crush pillar trial site (Figure 3.3) were stopped against a bracket pillar which was left along a 1.5 m strike orientated (E–W) reverse fault. No regional pillars were left in the trial section as two large potholes were situated either side of the trial site. The area above the trial site was supported by dip regional pillars which partially intersected the top portion of the trial area. Smaller potholes intersected in the mining area provided additional regional stability. By the end of February 2015, only one active mining panel remained in the trial site along which the final four crush pillars would be formed. Mining of the trial site was completed by the end of May 2015.

Table 3.1: Parameters of the trial site.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of mining (mbs)</td>
<td>782 - 800</td>
</tr>
<tr>
<td>Reef dip (deg)</td>
<td>8</td>
</tr>
<tr>
<td>Crush pillar width - planned (m)</td>
<td>2</td>
</tr>
<tr>
<td>Stoping width (m)</td>
<td>1.3</td>
</tr>
<tr>
<td>No. of pillars cut</td>
<td>55</td>
</tr>
<tr>
<td>Area mined (m$^2$)</td>
<td>22000</td>
</tr>
</tbody>
</table>

(arbitrarily chosen to include the entire crush pillar area)
Figure 3.3: Plan view indicating the trial area. The existing crush pillars are shown in red and planned pillars (P2, P3, P4) in green. The plan indicate the face positions during February 2015.

Of the 55 crush pillars cut, a range of actual width to height ratios were observed (Table 3.2). This was as a result of poor pillar cutting discipline as well as layout restrictions. In the trial area, approximately 22000 m$^2$ was mined. 87 Percent of the pillars had a w:h ratio of less than 2:1 and only seven pillars had w:h ratios greater than 2:1 with the maximum pillar width being 3.1 m.

Table 3.2: Pillar w:h ratio distribution at the trial site.

<table>
<thead>
<tr>
<th>w:h ratio</th>
<th>Percentage of pillars</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;1</td>
<td>29%</td>
</tr>
<tr>
<td>1 - 1.5</td>
<td>36%</td>
</tr>
<tr>
<td>1.5 - 2</td>
<td>22%</td>
</tr>
<tr>
<td>&gt;2</td>
<td>13%</td>
</tr>
</tbody>
</table>
Stress measurements conducted on K3 Shaft (Coetzer, 2003) at a depth of 820 mbs and in close proximity to the crush pillar trial site, indicated that the major principal stress was orientated horizontally (E-W). The measured virgin vertical stress (23.2 MPa) corresponds well with the calculated virgin vertical stress (24.1 MPa) at the corresponding depth. The k-ratio (ratio of horizontal : vertical stress) is approximately 1.8.

Figure 3.4: Virgin principal stress magnitudes and orientations from stress measurements conducted at K3 shaft.

At K3 shaft, the middling to the top of the Bastard Merensky Reef varies between 10 m and 13 m (Figure 3.5). The behaviour of the immediate hangingwall (HW1/HW2) is dominated by the overlying Bastard Merensky Reef as a result of the weak parting along the top and bottom contacts. Consequently, the hangingwall up to the Bastard Merensky Reef will act as deadweight on the support elements.

The upper Anorthosite series, which is found above the Bastard Merensky, is a competent zone. The support requirements for the immediate hangingwall (HW1, HW2, HW3) and upper Anorthosite series are therefore very different and must be catered for through the application of a crush and regional pillar system respectively.

Rock strength measurements determined the strength of the footwall Mottled Anorthosite is the strongest (236 MPa) compared to the reef (140 MPa), the hangingwall 1 Spotted Anorthosite (171 MPa) and hangingwall 2, Mottled Anorthosite (158 MPa).
The instrumentation programme during the trial was one of the most comprehensive monitoring exercises ever conducted at a crush pillar site in South Africa. The measured behaviour was related to the pillar condition as observed during the pillar forming cycle. This has not previously been recorded for crush pillars. It provided parameters to benchmark the performance of the pillar system against and can be used in future as a guide to identify acceptable pillar behaviour.
3.1. Visual observations

The condition of the pillars was visually assessed during underground visits. The objective was to determine if the pillars were in a crushed state, if the pillars were crushing while being formed at the face and if blocks of unmined ground (i.e. potholes) have an impact on the pillar behaviour.

Some of the general characteristics observed included:

- The composition of the pillar, including the exposed hangingwall rock, influenced the pillar behaviour.
- The fracture intensity was greater on the side of the pillar which was exposed first.
- Buckling of the fractured slabs was either along the top or the bottom of the pillar depending on the mining sequence. Pillars which fractured towards the west (Figure 3.6a) had the fractured slabs buckling closer to the footwall. In contrast, pillars which fractured towards the east (Figure 3.6b) had the fractured slabs buckling closer to the hangingwall.
- Pillars were observed to be partially fractured whilst being formed at the mining face (Figure 3.7). This indicated that the stress was sufficiently high to initiate early pillar crushing.
- Pillars that were completely failed contained fracture planes spaced approximately 30 - 50 mm apart (Figures 3.9 and 3.18).

![Figure 3.6a: Pillar scaling towards the west (mined first).](image-url)
Figure 3.6b: Pillar scaling towards the east (mined first).

Figure 3.7: Condition of pillar 37 whilst still part of the mining face. Refer to Figure 3.3 for the pillar position.
In the underground trial section, a distinctive failure pattern and failure sequence was identified in the pillars. Three modes of failure could be identified namely:

- Spalling (scaling),
- Buckling along failure or fracture planes,
- The formation of conjugate failure planes (wedge-like structure).

In Figure 3.8 these various failure modes can be seen. Many of the crush pillars in the trial site had the wedge-like structure forming where fractures intersected in approximately the centre of the pillar (Figure 3.9).

![Figure 3.8: Photograph of a crush pillar in the trial site. Visible in the photograph is the shearing of the fractured slabs along the hangingwall as well as the wedge formation in the pillar.](image-url)
Figure 3.9: Up-dip view of pillar P1. Note the wedge-like structure visible where the fractures intersect. Fractures clustered predominantly towards the east which was mined first.

During the extraction of the final mining section (Figure 3.3), three additional pillars were formed (P2, P3, P4). Two of these pillars were extensively instrumented. The following figures indicate the face position at a point in time relative to the pillar forming cycle. This was used as a reference to compare the observed external changes of a pillar to the measured behaviour.
Figure 3.10: Mining layout at the time when the trial site was temporarily stopped (July 2013).

Figure 3.11: Face position on 16 March 2015. Mining face aligned with pillar P2.
Figure 3.12: Face position on 14 April. Mining face aligned with pillar P3. Pillar P2 was now fully formed.

Figure 3.13: Face position on 23 April. Mining face aligned with pillar P4. Pillar P3 was now fully formed.
Figure 3.14: Face position on 6 May. Pillar P4 was now fully formed.

Figure 3.15: Face position on 18 May. All mining stopped.
The most prominent observations during the formation of the pillars in the final mining section were that; fractures formed parallel to the surface of the exposed abutment approximately 9 m ahead of the lagging face position. These fractures would increase as the lagging face advanced. By the time the face was aligned with the pillar, the fracture planes were spaced approximately 30 – 50 mm apart along the outer metre of the pillar (from the exposed surface of the pillar). These fractures would start to open (5 - 10 mm) as the pillar was being formed. Little to no scaling or slabbing was visible along the side of the pillar which was exposed by the advance of the lagging face. New fractures formed along this side of the pillar only when the pillar was 3 - 7 m in the back area. In most cases these new fractures were spaced more than 100 mm apart and extended to approximately half the pillar width. These fractures started to dilate when the pillar was approximately 10 m in the back area. It did not result in any observed scaling or slabbing of the pillar surface along this side of the pillar. Continued pillar deformation was observed until the pillar was approximately 14 m in the back area.

From the observations, a failure mode could be identified. This is illustrated in Figures 3.16 and 3.17.

- As a pillar is being cut, a “free face” is formed along the long axis of the pillar. Extension fractures form parallel to the exposed “free-face” and extend into the hanging-and footwall (Figure 3.16a).
- As new fractures form, previously formed fractures start to dilate (Figure 3.16b). The slabs defined by the fracture planes buckle at approximately mid-height as the slabs are pushed outward. This is observed as increased scaling towards the initially exposed side of the pillar (Figure 3.16c).
- The slabs move up-or down relative to one-another as the slabs rotate outwards while the pillar continues to dilate. The buckled slabs, in some instances, shear off along the hangingwall or along low angled extension fractures. As extension fractures continue to open and the fractured slabs migrate outwards, the confinement it provided is reduced. As a result more extension fractures are formed (Figure 3.16d).
- Where the extension fractures meet at approximately the centre of the pillar, the fractures and failure surfaces interact and form a wedge-like structure (Figure 3.17). Similarly, should the panel to the right be mined first the fracturing sequence will be concentrated towards the right side of the pillar.
Figure 3.16: Crush pillar fracturing sequence observed in the underground mining trial.
Figure 3.17: Crush pillar fracture pattern observed in failed pillars.

Figure 3.18: Failure patterns observed along the down-dip side of pillar 38 (1.5 m wide) illustrating the representation in Figure 3.17.
3.2. Measurements

At the end of June 2013, mining in the trial area was stopped to enable the required instrumentation to be installed and to control the future mining sequence. The mining layout at the time is shown in Figure 3.19. Mining resumed at the end of February 2015. The approximately 20 month stoppage was predominantly as a result of strike action in 2013 and 2014. Pillar P1, P2, P3 and P4 as indicated in the figures refer to the instrumented pillars.

![Diagram](image)

Figure 3.19: Plan view indicating the trial area and approximated pillar outlines. Pillars P2, P3 and P4 were formed during the mining of the final mining section as indicated by mining steps (Step 1 - Step 5).

The majority of mining took place in the E3 West panel (active instrumentation panel). Various types of instruments were used to achieve the desired objectives. Table 3.3 provides a summary of the instruments used (number in brackets refers to the number of instruments used). Figure 3.20 represents the instrumentation layout.
Table 3.3: Summary of the parameters measured and the equipment used.

<table>
<thead>
<tr>
<th>Measurement</th>
<th>Equipment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pillar stress</td>
<td>CSIRO Hollow inclusion (HI) cell (3) and Compact cononical-cone borehole overcore (CCBO) stress measurements (2)</td>
</tr>
<tr>
<td>Convergence in the panel</td>
<td>Continuous closure loggers (20), closure ride stations (33) and visual observations (impact on support)</td>
</tr>
<tr>
<td>Convergence next to the pillar to measure possible pillar deformation</td>
<td>Closure loggers and closure ride stations</td>
</tr>
<tr>
<td>Closure as a result of potential hangingwall separation along partings</td>
<td>Hangingwall extensometers (5)</td>
</tr>
<tr>
<td>Pillar dilation</td>
<td>Pillar extensometers (3), borehole camera surveys (3), pillar off-sets via pegs</td>
</tr>
<tr>
<td>Pillar fracturing</td>
<td>Borehole camera surveys, micro seismic probing (3 pillars), photogrammetry, visual observations</td>
</tr>
</tbody>
</table>

Figure 3.20: Plan of the actual instrumentation installed at the trial site.
3.2.1 Pillar stress

The stress change measured conducted approximately 4 m above each pillar was analysed by comparing the data to events corresponding to the various phases of the pillar forming cycle. Similar trends were identified for the three instrumented pillars. The results have revealed new information regarding the behaviour of crush pillars.

The data confirmed that a pillar reaches a residual state once the pillar is completely formed (holed). The pillars also experienced secondary reductions in stress when new pillars were formed. This is illustrated by the stress change measurement conducted above pillar P1 (Figure 3.21). The pillar was already completely formed prior to the extraction of the final mining section. The absolute stress measurement conducted at the end of 2014 indicated a vertical stress of 17.1 MPa. Underground, the pillar appeared to be in a crushed state (Figure 3.6b). However, when mining resumed in February 2015 until mining stopped in May 2015, there was another 10.58 MPa reduction in vertical stress. The most significant reductions appeared to coincide with the formation of new pillars (red dots highlighted in the figure). This phenomenon is believed to be associated with the transfer of stress to the newly formed pillars. This unloading phase has in the past typically only been referred to as continued strain softening behaviour.

Figure 3.21: Pillar P1 stress change versus time. The pillar was completely formed when the strain cell was installed.
The numerical model, used to back analyse the behaviour of the crush pillars in the trial section (refer to Section 4), was used to simulate the stress value at the point of measurement above each pillar. The model indicated a vertical stress of approximately 15 MPa (Figure 4.5) above pillar P1 (completely formed) for the layout which existed at the time of the measurement. This agreed well with the measured 17.1 MPa (Figure 3.21). Pillar P2 (Figure 3.22) demonstrated a reduction in vertical stress of 59.84 MPa, while for pillar P3 an overall reduction of 22.75 MPa was recorded.

The stress change data confirmed that the baseline magnitude obtained above the anticipated position of pillar P2 (18.2 MPa) was only relevant to the exact point of measurement. The model and stress change measurements indicated that the baseline vertical stress magnitudes of pillar P2 and pillar P3 should have been approximately 70 MPa (adjusted data in Figure 3.22) and 42 MPa respectively.

![Pillar P2 - Stress change measurement](image)

**Figure 3.22:** Pillar P2 stress change versus time. Adjusted data incorporates the total reduction in vertical stress. The strain cell was installed prior to the pillar being formed.

The continuous stress measurements indicated that:

- As the advancing face approaches a pillar to the point where it is aligned with the new pillar position, the pillar already experienced a significant reduction in vertical stress (23% - 64%).
• Whilst the pillar is being formed and until it is holed by the advancing face, the pillar experiences another reduction in vertical stress (28% - 41%). At this point the pillar reached its true residual state (14.8 - 23.74 MPa).

• As mining continues and new pillars are formed, the instrumented pillars continued to experience a reduction in vertical stress (38% - 41%). This secondary reduction resulted in a residual pillar stress of 6.5 - 17.4 MPa.

• The measured change in vertical stress as a result of the secondary stress reductions occurred within 6 m of additional face advance (after the pillar had been formed).

• Once the final pillar (P4) had been formed, there was no further change in vertical stress recorded above the instrumented pillars. The pillars nevertheless continued to deform.
3.2.2 Pillar fracturing

Visually a pillar is considered failed when the outer surfaces of a pillar bulges as failed fractured material dilates outward (scaling). It is generally assumed that even if the surrounding outer layers of a pillar are heavily fractured and the pillar may have reached a residual state, the core of a pillar is still solid. In the past this could only be investigated through the examination of holes drilled through a pillar. However, as indicated in Figure 3.23, these holes quickly close up once a pillar starts to fail. Also, new holes cannot be drilled through the fractured material. The inner portion of a pillar can therefore not be investigated.

Figure 3.23: Movement along the borehole drilled through pillar P3 from the western side. The hole could only be used for one scan before it was obstructed by the movement of the fractured slabs.

A new seismic velocity process was developed by the company IMS (Institute of Mine Seismology) and tested on the crush pillars in the trial site to determine the internal condition of the pillars. The process involves installing sensors and hitting points around the perimeter of a pillar at mid height. This allowed a two-dimensional horizontal plane passing through the entire pillar to be examined (Figure 3.24). The hitting points (pegs) are repeatedly struck with a hammer that has a seismic sensor attached. The recorded seismograms are used to determine the seismic travel time measured between the source and the respective sensors. The pillar outline is established using the surveyed co-ordinates of the installed pegs. High seismic velocities indicate zones of little or no fracturing. In contrast, low seismic velocities indicate zones of high fracturing.

The intervals of the scans were set out to determine:

- Fracture profile of the pillar before the pillar was formed (Stage 1); Baseline measurement.
- Fracture profile of the pillar when it has just been formed (Stage 2).
- Fracture profile of the pillar when in the back area (Stage 3).

An example of the result for pillar P3 is displayed in the subsequent figures.
Figure 3.24: Pillar micro seismic profiling method to determine the degree of fracturing experienced by a crush pillar (plan view).
The results indicated that the pillars experienced a significant overall reduction in seismic velocity once the pillars were formed (Stage 2). This compared well with the underground observations which showed a significant change in the pillar appearance at these times. Subsequent scans indicated that the pillars were already reasonably fractured during the pillar forming stage and that the advancing face only caused some damage to the eastern side of the pillar (lagging panel), although not visible along the exposed surface. The fracture or weakness planes therefore already existed along which dilation was taking place whilst the pillar was experiencing an unloading phase. The stress measurements confirmed that the pillars were already at a residual state when the pillars were completely formed (Stage 2).

Figure 3.25: Pillar P3 scan 1; pillar parallel with the face.
Figure 3.26: Pillar P3 scan 2; pillar formed. The pillar is 3 m behind the face position.

Figure 3.27: Pillar P3 scan 3. The pillar is 7 m behind the face position.
3.2.3 Convergence

Extensive convergence instrumentation was installed in the final mining section to understand the rock mass deformation and pillar behaviour as a function of mining and pillar crushing. Figure 3.28 indicates the approximate position of the closure stations.

Convergence was measured with both closure loggers and closure stations. All the continuous convergence data had a similar profile. The events leading to changes in convergence could be related to events in the mining cycle (pillar formation).

Figure 3.28: Layout of the closure stations in the final mining section.

Figure 3.29 illustrates the convergence measured in approximately the centre of the panel, in line with pillar P2. The results indicate that the formation of pillar P2 and P4 caused an increase in convergence of 6 mm and 3.5 mm respectively at this site. Similarly, the additional pillar deformation which continued to occur at this site once all the pillars were formed caused 3.5 mm of additional convergence.
Figure 3.29: Convergence measured at site C4.

Measurements conducted close to the mining face (e.g. Row E and F) indicated that a significant part of the convergence (approximately 15 mm) experienced in the stope occurred in the first 15 m behind the face (Figure 3.30). The measurements also indicated that the deformation of newly formed pillars contributed to the convergence experienced close to the stiff face abutment (e.g. P4). This demonstrates that pillar crushing does contribute to the convergence experienced close to the face.

The results of the closure stations installed either side of instrumented pillars indicated that the side of the pillar which was exposed first experienced the most convergence (related to fracturing sequence). The maximum amount of additional convergence measured was 32 mm. This was adjacent to the west side of pillar P2. The average rate of geometric convergence measured was 0.4 – 0.6 mm / m of advance. This could differ in other crush pillar sites and will be influenced by mining depth and regional stability.
Figure 3.30: Convergence measured as a function of face advance at site F4 installed adjacent to pillar P1. The estimated adjustment is to account for the convergence that occurs in the first 7 m behind the face before the instruments were installed.
4. BACK ANALYSES OF THE UNDERGROUND TRIAL

A limit equilibrium model implemented in the DD code TEXAN, was used to simulated the behaviour of the crush pillars. It is encouraging that the model appears to simulate the behaviour of the crush pillars observed in the underground trial section. The layout of the trial section was approximated using straight line polygons to enable the area to be easily discretised using triangular elements (Figure 4.1). The mining steps considered:

- Step 0: The layout with the face positions prior to the extraction of the final mining section.
- Step 4 and 5: Additional mining conducted to the final stopped face positions.

The element sizes selected for the mining steps were 1 m and the pillars 0.5 m.

Figure 4.1: Part of the meshing of the mined area, mining steps and pillars of interest.

Simulating the observed and measured behaviour experienced in the trial site was a significant challenge as modelling of a mine-wide tabular layout, which explicitly included a large number of crush pillars, has not been successfully performed in South Africa. The concept, initially tested by the author to determine the appropriateness of the method, proved successful (Du Plessis and Malan, 2011).
The intact and crushed material residual strength are mostly responsible for the onset of pillar failure and the residual stress state achieved by the model. Through successive cycles of parameter testing, the modelling parameters selected provided results which closely resembled the observed underground pillar behaviour (Table 4.1). Following this trial and error calibration of the model, both the observed and measured behaviour of the crush pillars in the trial site could be replicated.

Due to the flat dipping reef (8 degrees), a dip of zero degrees was included in the model to simplify the analysis. Also, the horizontal stress was assumed to be the same in both directions.

Table 4.1: Modelling parameters used for the back analysis.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young's Modulus (MPa)</td>
<td>70 000</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.25</td>
</tr>
<tr>
<td>Seam stiffness (MPa/m)</td>
<td>10^6</td>
</tr>
<tr>
<td>Dip (deg)</td>
<td>0</td>
</tr>
<tr>
<td>Mining depth (mbs)</td>
<td>782</td>
</tr>
<tr>
<td>Pillar height (m)</td>
<td>1.3</td>
</tr>
<tr>
<td>Vertical stress (MPa)</td>
<td>23.5</td>
</tr>
<tr>
<td>k-ratio</td>
<td>1.8</td>
</tr>
<tr>
<td>Intact material strength C₀ (MPa)</td>
<td>630</td>
</tr>
<tr>
<td>Crushed material strength Cₜ (MPa)</td>
<td>20</td>
</tr>
<tr>
<td>Intact slope (m₀)</td>
<td>8</td>
</tr>
<tr>
<td>Residual slope (mᵣ)</td>
<td>1</td>
</tr>
<tr>
<td>Friction angle (deg)</td>
<td>50</td>
</tr>
</tbody>
</table>

As a comparison, two constitutive models were used to simulate the reef material. The first simulation was for an elastic rock mass where the pillars were simulated as a “rigid” material that did not allow any failure or deformation. For the second simulation, the limit equilibrium constitutive model was used. This model allowed failure and load shedding of the pillars. Pillar P2 was selected to illustrate the evolution of pillar stress during the pillar forming process. Figure 4.2 indicates the average pillar stress (average of all collocation points within the pillar) for pillar P2 for each mining step for both the elastic and limit equilibrium model.

The pillar was initially part of the solid mining face abutment (Figure 4.3). The pillar is partially formed during mining step 1 and separated from the mining face during step 2.
Figure 4.2: Average pillar stress for pillar P2; Elastic versus limit equilibrium model. The pillar is holed in mining step 2.

Figure 4.4 considers a section a - a’ through pillar P2 (as indicated in Figure 4.3) and shows the evolution of pillar stress during each mining step using the limit equilibrium model to simulate the crush pillar behaviour.

- Step 0 considers the stress through the 2 m solid abutment at the pillar position.
- Step 1 indicates the stress through the pillar while the pillar is formed at the mining face.
- Step 2; the mining face advances and the pillar moves 6 m in to the back area.
Figure 4.3: Area indicating the final mining section extracted by mining steps 1 – 5.

Figure 4.4: Development of pillar stress during the pillar formation cycle (pillar P2); section a – a’ in Figure 4.3.
The results indicate:

- **Step 0:** High abutment stress along the exposed edge of the pillar.
- **Step 1:** Complete crushing of the outer 35 cm along the western side of the pillar. The pillar stress increases from the outer edges towards the centre of the pillar resulting in the high peaks indicated (57 cm from the western pillar edge). The core of the pillar, still presumably intact or in a partially crushed state is at a lower stress.
- **Step 2:** Complete failure of the pillar resulting in the residual state.

The behaviour predicted by the model for pillar P2 also coincides with the continuous stress measurement and pillar fracturing profile for this pillar. In both cases, the results indicated that the pillar was at a residual or failed state once the pillar was fully formed. The model therefore appears to be useful to simulate the pillar behaviour.

The model was used to evaluate the stress condition measured above the pillars (e.g. Figure 4.5). The model indicated a vertical stress of approximately 15 MPa above pillar P1 at the time of measurement which compared well with the measured 17.1 MPa. Similarly the model could be used to estimate the baseline vertical stress magnitudes above pillars P2 and P3.

![Vertical pillar stress (P1)](image)

**Figure 4.5:** Predicted vertical stress profile through a section of pillar P1 at the approximate absolute stress measurement position conducted 4 m above the reef.
An amount of convergence occurs close to the mining face before instruments can be installed (Figure 3.30). The model was used to simulate the convergence at these locations (e.g. site F4). The objective was to replicate the underground measurements and to understand the convergence profile. The model indicated that most of the convergence occurred very close to the face (14.45 mm in the first 2 m immediately behind the face). This is most likely a function of the limit equilibrium model which will also cause partial crushing of the face abutment. The estimated convergence (15 mm) based on measurement agreed well with the modelled prediction (13.95 mm). The model predicted a maximum convergence at the panel mid-spans of approximately 31 mm.

![Convergence versus face advance (F4) (Measured versus modelled results)](image)

Figure 4.6: Measured versus simulated convergence measured at site F4. The closure station was aligned with pillar P1 and was installed 7.7 m behind the face, prior to mining the final section.

Figure 4.7 illustrates how the model approximates the measured convergence at site D1 adjacent to pillar P3. This indicated that convergence alongside the pillars only started to occur once the pillars started to crush. The underground observations identified continued pillar deformation occurring in the back area of the stope. The measured convergence at the various sites also indicated ongoing convergence after mining had stopped. The majority of the additional convergence can therefore be attributed to ongoing pillar deformation.
Figure 4.7: Measured versus modelled convergence measured at site D1 adjacent to pillar P3. The pillar is formed in mining step 2 and holed in mining step 3.
5. CONCLUSION

As the South African platinum mines are increasing in depth, extraction ratios will decrease to unacceptable levels unless a crush pillar system is considered. Crush pillars are designed to crush when formed at the mining face. These failed pillars provide a local support function. However, some of the mines which currently use crush pillars, experience unpredictable pillar behaviour. This is especially problematic in the Merensky Reef (main platinum reef) and may result in pillar seismicity. With approximately 70% of the world’s platinum group metals being produced from these mines, it is imperative to understand crush pillar behaviour and the causes contributing to unpredictability to mitigate this risk.

The objective of the research was to determine the parameters which influence crush pillar behaviour to prevent the risk of pillar instability. In this study, various new methods to examine the behaviour of crush pillars were investigated. This resulted in an improved understanding of crush pillar behaviour.

A limit equilibrium model was identified as being able to simulate the behaviour of the pillars. As a novel contribution, the author derived an analytical model based on this approach to estimate the average pillar stress of a failed crush pillar. A limit equilibrium model implemented in the displacement discontinuity boundary element code TEXAN, was used to simulate the behaviour of the crush pillars. From the study, a set of design criteria could be derived as a guideline for the safe implementation of a crush pillar system. For the first time, parameters contributing to pillar instability could be investigated. The presence of unmined blocks of ground (geological losses, i.e. potholes) or poor mining layouts (i.e. oversized pillars), has up to now, not been considered in any analyses to evaluate crush pillar behaviour.

The behaviour of the crush pillars in a trial site established by the author, provided parameters against which the performance of future pillar systems can be benchmarked. From the observations at this site, the failure mode of the crush pillars could be recorded and interpreted. This has never been systematically recorded for crush pillars before and is an important research contribution. The instrumentation programme at this trial site was one of the most comprehensive monitoring exercises ever conducted at a crush pillar site in South Africa. The measured performance could be related to the pillar condition as observed during the pillar forming cycle. The results have revealed new information regarding the behaviour of crush pillars. Secondary unloading cycles, ongoing pillar dilation and continued convergence as a result of pillar deformation were identified. Also, the internal condition of pillars could be determined with a new seismic velocity process developed and tested on the crush pillars. The residual state of a crush pillar could therefore be validated and related to events in the pillar forming cycle.
The numerical analysis of the underground trial site validated the use of a limit equilibrium model to simulate the behaviour of crush pillars on a large scale. To date, no numerical modelling of a mine-wide tabular layout, which explicitly included a large number of crush pillars, had been reported in South Africa. This work is also a major contribution in this particular field. After the model was calibrated, both the observed and measured behaviour of the crush pillars in the trial site could be replicated. The study illustrated that the simulation of an idealised layout is nevertheless valuable compared to a large scale underground layout. It is easy to set up and with little effort, trends can be established through consecutive runs. Design guidelines derived from the preliminary modelling results were compared and could be verified by observed behaviour and actual measurements in the trial mining site.

The research findings improved the industry’s understanding of crush pillar behaviour. The key outcomes will guide mines on the safe implementation of a crush pillar system. This includes strategic considerations such as; defining the depth at which a crush pillar system could be considered, the impact of geological losses in an ore reserve on the pillar behaviour, the ideal pillar size required to ensure effective pillar crushing and preferred stope layouts to avoid unpredictable pillar behaviour or the formation of oversized pillars. This work therefore has the potential to have a significant impact on the profitability and sustainability of the platinum industry in South Africa.
6. REFERENCES


