Safety in Mines Research Advisory Committee

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THE DETERMINATION OF LOADING CONDITIONS FOR CRUSH PILLARS AND THE PERFORMANCE OF CRUSH PILLARS UNDER DYNAMIC LOADING

Final Report

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Executive summary

The objective of this work was to develop a design methodology for crush pillars under different loading conditions in South Africa’s gold and platinum mines. To achieve this, the variations in crush pillar characteristics were determined with a survey of current crush pillar design and actual geometries. This was complemented by extensive underground investigations and monitoring of the environments (i.e. the hangingwall, the reef and the footwall) in which crush pillars were deployed.

Concurrent with these underground studies the dynamic behaviour of crush pillars in response to seismic events was investigated, finding no explicit correlation between the occurrence of distal events and the seismicity within a crush pillar. Also, micro-seismicity associated with brittle fracturing of the rock was recorded for some distance beyond the point at which the pillar should have been formed and fully crushed. These findings, which did not agree with accepted ideas on crush pillar behaviour, resulted in a shift of focus of the project to see what the behaviour of crush pillars was underground.

Detailed, time dependent studies were conducted and the Pillar Fracturing Index (PFI) was developed. From the underground studies it became clear that the behaviour of crush pillars was not truly understood, and a methodology for modelling crush pillars that mimicked underground conditions had to be developed. Intensive UDEC modelling was used to define the actual rock mass characteristics of a crush pillar. Using the underground measurements, a simple constitutive model of a discontinuous blocky rock mass controlled by a simple Mohr-Coulomb failure model was developed. The PFI measurements allowed the quantification of the numerical experiments. These studies show that due to the variety of geotechnical and mining conditions it is unrealistic to develop design charts, as too many assumptions and special cases would have to be included, preventing the development of a broadly applicable and practical tool. To assess the behaviour of different crush pillars under different mining and geotechnical conditions, the PFI must be measured. This quantification gives a measure of the pillar’s condition relative to other pillars in similar geotechnical or mining environments. The numerical modelling approach outlined here can be used to qualitatively understand the actual environmental conditions for the pillars. It should however be remembered that these models can only be fully quantifiable with sufficient input data (including PFI measurements) and model experiments.

The PFI and numerical models combined provide a practical tool for determination of a pillar’s condition and performance.
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1 Survey of Current Crush Pillar Practice

1.1 Introduction

A ‘pillar’ is defined as any block of unmined ground surrounded by mining. The functions of pillars are for hangingwall/roof support or regional control. Typically, non-yield, yield and crush pillars fall into the category of hangingwall/roof support; whilst barrier pillars, water and boundary pillars, stabilizing pillars, bracket pillars and shaft pillars function as regional control pillars.

In general, pillars are expected to carry their design loads for indefinite periods. Pillars have been used extensively since the early days of mining. The pioneers of the use of hard-rock pillars in South Africa were gold mines which operated at relatively shallow depths. As workings extended deeper, many problems associated with pillars were noticed, including pillar bursting. This was due mainly to incorrect design, as there was little understanding of pillar behaviour and their interaction with the rock mass loading the pillars (Ozbay et al., 1995). Pillars have been used as stope support in Bushveld Complex (BC) mines from the start of mining (Ozbay et al., 1995). They remain the main support component in most present-day BC mines.

Considerable experience has been accumulated over the years on designing pillar layouts. However, there are still major concerns, not the least of which is mining to greater depths. Furthermore, in cases where the evolved methods provide the required safety, the question of whether the optimum extraction is being achieved remains unanswered (Ozbay et al., 1995).

The optimal design of pillar layouts requires an improved understanding of the mechanistic behaviour of pillar material, and of the interaction between pillars and the surrounding strata. The mechanisms involved are often complex and generalised formulations may not be applicable to all cases. There are many factors which can affect pillar performance in different environments, such as cohesion, internal friction, contact friction, discontinuities, stress state, panel dimensions and extraction ratio. There have been a number of studies to estimate the impact of these factors. However, to take all these factors into account in design is time-consuming and expensive. Another way is to conduct back analyses of intact and collapsed pillar cases. This was achieved in coal mines in 1967 after the Coalbrook disaster (Salamon and Munro, 1967), which produced the well known Salamon & Munro coal pillar strength formula. The most important advantage of this formula is that because the data was collected from many different geotechnical environments, it takes into account the impact of all relevant factors. It is therefore suggested that a pillar design/evaluation tool should be developed which determines the performance of hard-rock pillars under the actual environmental conditions to which they are subjected.
1.1.1 Definition of crush pillars

Today, pillars are used as an integral part of the support strategies of all gold, platinum and coal mines; from very shallow (<20 m) to very deep workings (>2500 m). Mining requirements dictate the function and nature of these pillars, including stabilization, yielding or crushing. The stress-strain curve of a pillar is shown conceptually in Figure 1-1. The initial straight line portion up to the yield point corresponds to the elastic response of the pillar, and the slope of this portion of the curve is the effective Young’s Modulus of the pillar. The yield point indicates the onset of localised inelastic behaviour, or failure of some material in the pillar. After the yield point, the pillar exhibits strain hardening until the peak strength is reached. Load shedding then occurs, until a residual strength is reached.

![Stress-strain curve diagram](image)

Figure 1-1: A qualitative diagram of the complete stress-strain curve of a pillar (after Ryder & Jager 2002).

Different overall pillar behaviours may be expected with different ratios of pillar width to pillar height (w/h). The range of possible pillar stress-strain behaviours is shown in Figure 1-2. Pillars of low w/h ratio (up to about 4) tend to exhibit a well-defined peak stress $\sigma_s$, and the post-peak slope of the curve can be steep, roughly the same order of magnitude as the Young’s modulus (though with a negative sign).

As the w/h ratio increases, the yield stress increases, the strength increases, and the slope of the post peak portion of the curve becomes flatter. At a w/h ratio equal to about 5, the post peak slope may become approximately flat, but brittle (unstable) failure, similar to that experienced by more slender pillars, can continue to occur near the edge of the pillar.

At large values of w/h ratios (greater than 6 or 7), pillars tend to strain harden after the yield point. These pillars are termed ‘squat pillars’, and the yield stress increases considerably. However, the commonly held belief that pillars of w/h = 10 are ‘indestructible’ is not entirely true. Pyroxenite and coal samples have been tested to destruction at w/h = 10 in the laboratory.
Underground monitoring of stabilizing pillars of w/h $\approx 20$ in a deep gold mine showed that fracturing occurred throughout the width of the pillar; though this is not to say that the pillar had necessarily failed completely. In addition, in most shallow-mining applications, the stresses applied to pillars of w/h $> 10$ are well within the capacity of the pillar. Figure 1-2 illustrates the stress-strain behaviour of various pillar types, showing the expected behaviours for different ranges of pillar w/h ratios.

![Stress-strain behaviour of pillar types](image)

Figure 1-2: A qualitative diagram of the stress-strain behaviour of pillar types of different w/h ratios (after Jager & Ryder 1999). The term “elastic pillar” applies to any pillar designed not to be loaded beyond its yield point.

These different pillar types are usually used in the following layouts:

Elastic (non-yield) pillars are intended to remain essentially unfailed and elastic during the life of the mine. They are usually employed in bord and pillar panels. The w/h ratios are on occasions as low as 0.7 at very shallow depths, but are more usually 4 and more. Design safety factor is typically $> 1.6$.

Crush pillars which are intended to ‘crush’ while they are still part of the face and to have already reached their residual strength when cut, so that the pillars can substantially yield at their residual strength. The w/h ratios are typically 1.7 to 2.5.
Yield pillars which are intended to have SF > 1 when first formed, but to then yield in a stable manner at residual stress levels of about peak strength. The w/h ratios are as low as 3 but often approach 5.

Barrier pillars are usually employed between the panels to ensure regional stability. These pillars are intended to remain unfailed and elastic during the life of mine, with w/h ratios usually of the order of 10.

The present study focuses on the in situ performance of crush pillars, with some attention also to the behaviour of yield pillars, as there is essentially a continuum in behaviour between these two pillar types.

### 1.1.2 Actual crush pillar design and dimensions

As part of SIMRAC and other projects, many reviews of hard-rock pillars have been conducted in the past (GAP 027, GAP 028, GAP 334, GAP 617, PLATMINE Task 1.2). Another complete literature review was therefore not conducted for this project, but the following points are worth mentioning.

Crush pillars are usually in a post-failure state, i.e. the peak strength has been exceeded and the residual strength of the pillar has been attained. Currently, there is no scientifically proven technique or methodology for crush pillar design in South African mines. The dimensions of crush pillars are generally based on experience, with pillar width to mining height (w/h) ratios less than 2.5. The lower limit of w/h ratio is governed by practical mining considerations, and is typically about 2.0 for stoping widths of up to about 1.5 m. It has also been suggested that the pillar minimum width should not be less than 2.0 m. In practice, many mines use inherited pillar dimensions, often adapted from other (usually neighbouring) mines.

Crush pillars, in theory, are able to support much less load than elastic/non-yield pillars, and therefore permit much higher levels of regional closure. Thus it is suggested in the literature that crush pillars should best be used in conjunction with regional pillars. The ultimate function of crush pillars is to support the tensile zone between regional pillars, or to provide support of the deadweight up to any well-defined (stratigraphic or structural) partings in the hangingwall. It is important to note that the current design of crush pillars in the industry is limited to the given minimum and maximum w/h ratios and pillar widths listed in the previous paragraph. However, many so-called crush pillars are not subjected to early crushing and are therefore acting as under-designed elastic pillars with sometimes serious safety implications. When a so-called crush pillar is too wide it may fail violently instead of gradually; or because of its high strength may shear the hangingwall around the pillar corners; or create problematic footwall heaving. Conversely, if the pillars are too small, they may not be able to supply sufficient residual support resistance and may fail completely permitting high stope deformations.

Nevertheless, it is well known that the above simple design guidelines have successfully been used in many gold and platinum mines over many years in South Africa. This obviously raises the question of why some of those pillars designed using these limited design guidelines worked so well. This is probably because they were formed in similar macro environments where the
tensile zone of the regional spans was in most cases limited. Due to pillars formed as a result of geological irregularities in platinum mines (especially potholes), and specifically designed regional stability pillars in gold mines, the spans between the regional pillars are usually limited to 400 to 500 m, which result in a ‘bridging’ effect. Generally, the stoping widths are also in a relatively tight range, from 1.0 m to 2.0 m. Oversized so-called crush pillars are much more dangerous than proper crush pillars due to the risk of pillar bursting. Therefore, it is suggested that oversized crush pillars should be avoided as much as possible.

In order to quantify the quality of dimension control in the cutting of crush pillars, a detailed study was conducted in a platinum mine, which previously had bursting problems and associated injuries and fatalities. This study took place on a mine in the Western Limb of the Bushveld Complex, with three main shafts.

At Shaft A, two different crush pillar design dimensions are used, namely 3 x 3 m pillars and 4 x 4 m pillars. While 4 x 4 m crush pillars are used without sidings, 3 x 3 m pillars are used with sidings. The stoping width is approximately 1 m. Altogether, 241 pillars were measured (width and length) by the mine personnel at Shaft A for a period of 6 months. The deviations of pillar widths from the original design are shown Figure 1-3. Similarly, Figure 1-4 shows the deviations of measured pillar length from the design dimensions. Table 1-1 summarises the percentage of undersize, correct size (within > 0.5 m) and oversize pillars at Shaft A.

![Figure 1-3: Pillar width deviations from the design dimension, Shaft A.](image-url)
As can be seen from this table and the above figures, pillar dimension control is poor and the majority of the pillars are cut oversize. Less than 18 % of all 241 pillars are close to the correct size.

Two different crush pillar design dimensions are used at Shaft B, namely 4 x 4 m pillars, and 4 x 3 m pillars. While 4 x 3 m pillars are used in conjunction with sidings, 4 x 4 m pillars are used without sidings. Altogether, 516 pillar dimensions, measured between May 2004 and October 2004, were obtained from mining personnel. The deviations of pillar dimensions from the original design are shown in Figure 1-5 and Figure 1-6.
Figure 1-5: Pillar width deviations from the design dimension, Shaft B.

Figure 1-6: Pillar length deviations from the design dimension, Shaft B.

Table 1-2 shows the percentage variation from the design dimensions. Although it appears that pillar dimension control is better at Shaft B compared to Shaft A, 28% of pillar widths are oversize (and almost 60% of pillar lengths are greater than the design dimension, with implications on ventilation control).

Table 1-2: Summary of pillar dimensions at Shaft B.

<table>
<thead>
<tr>
<th>Deviation from design (%)</th>
<th>Pillar width</th>
<th>Pillar length</th>
</tr>
</thead>
<tbody>
<tr>
<td>Undersize</td>
<td>38.6</td>
<td>18.4</td>
</tr>
<tr>
<td>Correct size</td>
<td>33.1</td>
<td>22.3</td>
</tr>
<tr>
<td>Oversize</td>
<td>28.3</td>
<td>59.3</td>
</tr>
</tbody>
</table>
Shaft C uses 4 x 4 m pillar and 4 x 3 m pillar crush pillar design dimensions similar to those at Shaft B. A total of 178 pillar dimensions were analysed from Shaft C. The deviations of pillar widths and lengths from the original design are shown in Figure 1-7 and Figure 1-8, respectively. Similar to Shafts A and B, pillar dimension control is a serious problem at this shaft (see Table 1-1).

**Figure 1-7: Pillar width deviations from the design dimension, Shaft C.**

**Figure 1-8: Pillar length deviations from the design dimension, Shaft C.**
Table 1-3: Summary of pillar dimensions at Shaft C.

<table>
<thead>
<tr>
<th>Deviation from design (%)</th>
<th>Pillar width</th>
<th>Pillar length</th>
</tr>
</thead>
<tbody>
<tr>
<td>Undersize</td>
<td>30.3</td>
<td>9.0</td>
</tr>
<tr>
<td>Correct size</td>
<td>32.6</td>
<td>19.7</td>
</tr>
<tr>
<td>Oversize</td>
<td>37.1</td>
<td>71.4</td>
</tr>
</tbody>
</table>

1.1.3 Pillar width to height ratios

As mentioned previously, crush pillar design is based on the width to height ratio. It is commonly accepted that pillars of w/h ratio of 3 - 5 potentially fail in a violent manner, and it is usually recommended that crush pillars be designed with a w/h ratio less than 3.0. The lower limit of w/h ratio is usually defined by practical mining considerations, and is set at about 2 for stoping widths of up to about 1.5 m (Ryder & Jager, 2002).

As a quality control check, the mine conducted a total of 264 pillar and stoping width measurements at Shafts A and C. Figure 1-9 shows the frequency and cumulative percentage frequency distributions of measured pillar width to height ratios. This figure indicates that 86 % of all pillars have w/h ratio greater than 2.5, and 52 % of all pillars have w/h ratio greater than 3.0. There is therefore a strong possibility of pillar bursting in areas where the pillars are not crushed, especially at Shaft A, due to the very competent hanging wall.
Figure 1-9: Crush pillar width to height ratios at Shafts A and C.

1.1.4 Currently used Crush Pillars

The behaviour of pillars in gold and platinum mines has been discussed by many authors in the past. Crush pillars are interior pillars which are designed to support a mining panel. A typical scattered mining layout with crush pillars is shown in Figure 1-10. Here, the crush pillars are cut on the down-dip side of strike gullies and holings are provided for ventilation. This layout is fairly typical of both gold and platinum hard rock tabular mines in South Africa, although the potholes illustrated are restricted to the BC (platinum and chrome mining).
Many gold and platinum mines use crush pillars with strike dimensions of 2, 3, 4 or 6 m and dip dimensions of 2 or 3 m, separated by 0.5 to 3 m wide ventilation holings. These pillars are normally located on strike and with a 1 m siding on the down dip side of the strike gully. During the course of this study it was noticed that many mines today call the same size pillars ‘crush’ or ‘yield’ pillars interchangeably, at similar depths.

1.1.5 Currently used Yield Pillars

So-called yield pillars are supposedly intact when cut but are then expected to yield in a stable manner in the back areas at a later stage. Pillars with w/h of approximately 4 to 5 are currently thought to provide a flat or positive post-peak slope and can be safely loaded beyond their elastic limit. For example, some mines operating in the UG2 reef at a depth of about 300 m use 5 x 5 m yield pillars located along strike with 36 m centre spacing.

Existing theoretical knowledge on yield pillar design predicts possible stability only when the post-failure pillar modulus is less than that of the surrounding strata. Coal mining practice confirms this finding, since signs of stable pillar failure are observed in coal mines having post-
failure modulus $E_s \approx -0.4 \, E$, as reported by Oldroyd & Buddery (1988). At present, the design of hard rock yield pillars is largely empirical. Little is known of the fundamental behaviour of yield pillars, and inconsistencies are often evident between theory and practice. Theoretical studies indicate that the levels of regional or local stiffness do not favour stability for relatively slender hard rock pillars, yet preliminary observations indicate stable load shedding of these pillars at $w/h$ ratios of less than 2.5 (Noble 1993, Lougher 1993). Also, it was thought that pillars with $w/h \geq 5$ cannot fail in an unstable manner, but ‘bursting’ of 5 x 5 m pillars in the back areas was reported by Spencer and Kotze (1990). These cases need to be rationally analysed using numerical and laboratory modelling, together with the collection of further in situ data, as in the studies given in the following sections of this report.

1.2 Survey conclusions

There is a serious lack of discipline of cutting correct pillar sizes. This can, in principle and also in practice, result in pillar bursting problems and therefore it is recommended that the sizes of crush pillars should be regularly checked and reported to Rock Engineering Departments for necessary remedial action.
2 Underground monitoring

2.1 Introduction

The rock mass and mining conditions of forty different sites at nine different shafts have been quantified. In addition, detailed geotechnical monitoring has taken place at eleven sites. Data from closure-ride stations, extensometers, borehole camera surveys, geological mapping, geotechnical mapping and seismic monitoring has been collected.

Pillars are part of a complex system including the hanging and footwall. Failure is focused on the weakest part of the system, and it was thus necessary to monitor the entire system and not just the pillar itself. This was achieved via the determination of depth, mining history, rock types and their distribution as well as geotechnical and failure characteristics. The strengths and thickness of the immediate hanging and footwall rock types were determined. Characteristics of the rock mass of the pillars and surrounding strata were defined by fracture mapping, the quantification of failure mechanisms and mapping of time-dependent changes.

The deformation characteristics of the pillars and surrounding strata were quantified via borehole camera observations, closure-ride and extensometer measurements. A novel method for quantifying pillar behaviour that takes into account all of the complex interactions in this system has been developed. This is termed the Pillar Fracture Index (PFI) and was determined for the various underground monitoring sites. Section 2.4 describes the methodology and uses of the PFI in detail. Table 2-1 indicates the sites investigated and instrumented for detailed monitoring. The table shows that a wide range of gold and platinum sites were visited and monitored.
Table 2-2 defines the type of information gathered at each site. Details of these observations at the different underground sites are described in the following sections.

**Table 2-1: Underground monitoring activities.**

<table>
<thead>
<tr>
<th></th>
<th>Investigations</th>
<th>Instrumentation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gold Mine A</td>
<td>4 geotechnical</td>
<td>2 geotechnical, 1 seismic</td>
</tr>
<tr>
<td>Gold Mine B</td>
<td>4 geotechnical</td>
<td>None</td>
</tr>
<tr>
<td>Gold Mine C</td>
<td>2 geotechnical</td>
<td>1 geotechnical</td>
</tr>
<tr>
<td>Platinum Mine A</td>
<td>2 geotechnical</td>
<td>1 geotechnical</td>
</tr>
<tr>
<td>Platinum Mine B</td>
<td>4 geotechnical</td>
<td>3 geotechnical</td>
</tr>
<tr>
<td>Platinum Mine C</td>
<td>1 geotechnical</td>
<td>1 geotechnical</td>
</tr>
<tr>
<td>Platinum Mine D</td>
<td>19 geotechnical</td>
<td>None</td>
</tr>
<tr>
<td>Platinum Mine E</td>
<td>Detailed previous work</td>
<td>1 detailed geotechnical</td>
</tr>
<tr>
<td>Platinum Mine F</td>
<td>1 geotechnical</td>
<td>None</td>
</tr>
<tr>
<td>Mine</td>
<td>Site No.</td>
<td>Monitor/visit</td>
</tr>
<tr>
<td>-------------</td>
<td>----------</td>
<td>---------------</td>
</tr>
<tr>
<td>Gold Mine A</td>
<td>1</td>
<td>V Y Y Y Y Y Y</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>M Y Y Y Y Y Y</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>M Y Y Y Y Y Y</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>V Y Y Y Y Y Y</td>
</tr>
<tr>
<td>Gold Mine B</td>
<td>1</td>
<td>V Y Y Y Y Y Y</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>V Y Y Y Y Y Y</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>V Y Y Y Y Y Y</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>V Y Y Y Y Y Y</td>
</tr>
<tr>
<td>Gold Mine C</td>
<td>1</td>
<td>M Y Y Y Y Y Y</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>M Y Y Y Y Y Y</td>
</tr>
<tr>
<td>Platinum Mine A</td>
<td>1</td>
<td>M Y Y Y Y Y Y</td>
</tr>
<tr>
<td>Platinum Mine B</td>
<td>1</td>
<td>M Y Y Y Y Y Y</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>M Y Y Y Y Y Y</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>M Y Y Y Y Y Y</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>M Y Y Y Y Y Y</td>
</tr>
<tr>
<td>Platinum Mine C</td>
<td>1</td>
<td>V Y Y Y Y Y Y</td>
</tr>
<tr>
<td>Platinum Mine D</td>
<td></td>
<td>V Y Y Y Y Y Y</td>
</tr>
<tr>
<td>Platinum Mine E</td>
<td>1</td>
<td>M Y Y Y Y Y Y</td>
</tr>
<tr>
<td>Platinum Mine F</td>
<td>1</td>
<td>V Y Y Y Y Y Y</td>
</tr>
</tbody>
</table>
2.2 Underground Sites

2.2.1 Gold Mine A

At Gold Mine A, four different sites were investigated and instrumented to different levels of detail. Site 1 was mapped, Site 2 was mapped and instrumented and Site 4 was mapped and instrumented and seismically monitored (see Section 2.3). Unfortunately Site 3 had to be abandoned shortly after it was established. At the sites investigated and monitored mining is taking place on the VCR at depths between 1600 and 2050 m below surface. The mining is in a breast configuration with 3 x 2 m in-stope pillars spaced 15 m apart on strike and 20 m on dip. The stoping width varies between 1.75 and 2.1 m and stopes are about 120 m apart.

2.2.1.1 Gold Mine A – Site 1

At this site, three discontinuity sets were mapped, including the mining induced fractures. The calcite-filled geological joints usually bound the edges of the fall of grounds (FOGs). In the pillar, the mining induced fractures (MIFs) are closely spaced (10 cm apart) and dip steeply (80 degrees) away from the centre of the pillar. Outside the pillar, the steeply dipping MIFs are restricted to the immediate hangingwall (i.e. the tuff), where they dip towards the centre of the pillar at approximately 65 °. Where damage has occurred to the hangingwall (due to the fracturing of the tuff), large falls of ground can take place, which extend laterally to the calcite joints. Otherwise, the top reef contact is sharp and the MIFs do not extend into the lava hangingwall. Some shearing is evident along the contact. The FOG of the tuff also changes the width to height ratio of the pillars making them taller, which may affect the pillar behaviour (Figure 2-1). Where footwall heave does occur, it appears to be restricted to the first 40 cm, possibly terminating on a bedding plane.

At the face, the mining induced fracturing continued for a depth of at least 50 cm into the face and the subsequent pillar. These MIFs were spaced approximately 15 to 20 cm apart. In contrast to this, the steep dipping MIFs in the footwall of the face area were spaced less than 10 cm apart, which suggests that the footwall is the most highly stressed element in the hangingwall - pillar - footwall system.
2.2.1.2 Gold Mine A – Site 2

Unlike the previous site, no tuff was evident in the hangingwall at this site (Figure 2-2). The contact between the hangingwall and the reef is however marked by a non-persistent yellow mylonite fault, which appears to prevent the MIFs present in the reef from penetrating into the hanging. Low angle (8° dip) fractures are present in the hangingwall (Figure 2-3). Geotechnical instrumentation was installed at this site (Figure 2-4) in anticipation of a crush pillar being cut, but this did not occur.

In addition to the intermittent mylonite fault along the top reef contact, there was a dominant quartz-vein joint set oriented 20° down dip of strike with a steep dip of 75°. The conjugate set
of the low angle hangingwall MIFs dip steeply into the face in the reef and about 10 cm from the bottom reef contact (BRC). The MIFs then change orientation to dip away from the face at about 65° (Figure 2-5). These fractures form planes along which some footwall heave occurred.

Figure 2-3: Closure-ride station 1 - note hangingwall dominated by low angle MIFs. Figure 2-4 indicates position of this station.

Figure 2-4: Plan showing geological and geotechnical features of Site 4, as well as instrument positions.
The fracturing in two drill-holes was examined using a borehole camera (Figure 2-6). These results are presented graphically in Figure 2-7 and were used to determine Pillar Fracture Indices. The Pillar Fracture Index (PFI) concept is explained in Section 2.4. A PFI of 12 and 29 was determined for Holes 1 and 2 respectively, which is similar to other PFIs ascertained for crush pillars. The lower PFI in Hole 1 is because it did not penetrate as deep and hence was only within the crushed skin of the excavation, rather than taking into account all of the rock mass that would make up the pillar. The average spacing of fractures in Holes 1 and 2 was very similar at 0.05 and 0.06 m, suggesting that merely taking this average does not allow discrimination of subtly different rock mass conditions at varying distances from the skin of the excavation.
Figure 2-6: Borehole camera in observation hole (Hole 2) at the centre of the planned pillar.

Figure 2-7: Fracture spacing within the two observation holes in the planned pillar position.

Closure ride information has been collected from this site and is shown in the graphs below (Figure 2-8). Closure appears to be relatively constant across the stations at approximately 40 mm per week, with variable dip and strike ride. Due to adverse ground conditions (including a major fall of ground), only three closure-ride stations were installed and monitored.
2.2.1.3 Gold Mine A – Site 3

Instrumentation, including numerous closure-ride stations, borehole camera monitoring holes and extensometer holes, was planned for the site, but as the cross-cut access to this site collapsed, it was abandoned. The geological and geotechnical conditions are thus only briefly described. Mining was occurring on the Ventersdorp Contact Reef, with a shale footwall and competent lava hangingwall. There were small patches of tuffaceous lava in the immediate hangingwall, but the pillar rock mass behaviour was dominated by movement of the softer shale footwall. The mining induced fracturing (MIF) in the pillars was steep (approximately 80°) and continued for a depth of at least 50 cm into the face and the subsequent pillar. These MIFs were spaced approximately 15 to 20 cm apart. The MIFs in the footwall were closely spaced at less than 10 cm apart and steeply dipping.

2.2.1.4 Gold Mine A – Site 4

This site was selected so that seismic monitoring of a crush pillar could be undertaken. Detailed geotechnical mapping and instrumentation were done to complement this seismic monitoring. The seismic monitoring is reported on in Section 2.3. At this site the footwall consists of clean quartzite, with the channelised, sand-rich conglomerate of the VCR overlying it. The
hangingwall consists of a 30 to 50 cm band of highly sheared tuffaceous material, before becoming fine-grained lava (Figure 2-9). This tuffaceous material often falls out, resulting in pillars that are of a width to height ratio smaller than design.

Figure 2-9: View of the hangingwall showing the large amount of fall-out of sheared, tuffaceous lava above the top-reef contact.

Fracturing in the lava hangingwall was low angle (>20 º), and dipped towards the face. This is enhanced by the bedding parallel shearing in the immediate hangingwall. In the weaker hangingwall tuff, the fracture density almost doubles from 30 fractures per meter in the reef to almost 70 per meter (Table 2-3). The fracture frequency in the hangingwall lava is reduced by horizontal movement along the flat dipping shears. Figure 2-10 indicates the strengths of the different stratigraphic units. The higher fracture frequency in the equally strong reef may also be due to low cohesion contacts on the hanging and footwall, allowing dilation of the pillar.

Table 2-3: Fracture characteristic at Gold Mine A, Site 3.

<table>
<thead>
<tr>
<th>Position</th>
<th>Rock Type</th>
<th>Fracture</th>
<th>Fracture</th>
<th>Strike</th>
<th>Dip</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Frequency</td>
<td>Strike</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hangingwall</td>
<td>Lava (APF)</td>
<td>10 / m</td>
<td>180</td>
<td>17</td>
<td></td>
</tr>
<tr>
<td>Stope face</td>
<td>Tuff</td>
<td>70 / m</td>
<td>5 or 95</td>
<td>70</td>
<td></td>
</tr>
</tbody>
</table>


<table>
<thead>
<tr>
<th>Stope face</th>
<th>Conglomerate (VCR)</th>
<th>30 / m</th>
<th>5 or 95</th>
<th>65</th>
</tr>
</thead>
<tbody>
<tr>
<td>Footwall</td>
<td>Quartzite</td>
<td>8 / m</td>
<td>150</td>
<td>30</td>
</tr>
</tbody>
</table>

180 - 300 (ave. 250) MPa

Alberton Porphyry Formation (APF) lava

Tuffaceous quartzite (APF)

180 - 300 (ave. 250) MPa

Ventersdorp Contact Reef conglomerate

170 MPa

Quartzite

Figure 2-10: **Stratigraphy and rock strengths associated with the crush-pillars at Gold Mine A.**

The dip of the fractures is very similar at 65 and 70 ° in the reef and tuff respectively. The contact between the tuff and the overlying lavas has a very low cohesion, revealed where the hangingwall is unsupported the fractured tuff falls out exposing low angle fractures (17 ° dip) in the lava (Figure 2-11 and Figure 2-12).

The mining induced fractures in the stope-face are oriented parallel to the face and, depending on whether the mining-face or the pillar edge is examined, their strike varies by approximately 90 °. This suggests that fracturing does not occur ahead of the face, during face advance, but after the pillar is cut. The fracture frequency in the lava hangingwall at this site is similar to that measured at Site 1, where the pillar was yet to be cut. The fracture frequency in the conglomerate and tuff is five to ten times more than at Site 1, indicating that much of the fracturing occurred after the pillar was cut.
**Figure 2-11:** Photograph showing the abundance of steep fracturing of the reef and overlying tuff.

**Figure 2-12:** Footwall conditions of the pillar shown in

**Figure 2-11.**

Footwall heave is not as apparent as with the other sites, but some opening is evident on the discontinuities in the footwall (up to 5 mm). This may be reduced because of the low angle of the fractures here (30 °), which causes the slabs to close up on one another. The higher fracture frequency in the reef suggests that it is definitely the pillars that are failing and not the pillar punching into the weaker footwall. The high pillar fracture frequency meant that the closure measured at this site was, as expected, slightly higher, reaching 50 mm per week. There is also less ride (both dip and strike) at this site.
2.2.2 Gold Mine B

At this mine, no monitoring sites were established; rather details of the geological, mining and geotechnical characteristics of the crush pillars and the rock mass in the stope around them were captured. At this mine, breast and scattered mining with 10 x 10 m in-stope pillars spaced 25 m apart is taking place. However, due to the age and extent of the mining as well as extensive geological discontinuities, there are considerable variations in this pattern. As a result the stope span also varies considerably from about 30 m to over 100 m.

2.2.2.1 Gold Mine B – Site 1

Five different pillars were examined in detail at this site. The pillar size was nominally 10 by 10 m. The general geology was noted to determine how the various pillars were responding as a result of both changes in the geology and mining history. At this site, the reef dips at 13 °. Two orthogonal joint-sets are present, dipping at 88 ° and 70 °. The first lies 30 ° up dip of strike and the second 10 ° up dip of reef dip. No mineralization is present on these joints. The sedimentary package at this site is an upward fining erosion-channel fill sequence of sediments (Figure 2-13). In this area a thin (10 cm) shale band is present between the top reef contact and the overlying gritty light grey quartzite. The reef is a small pebble conglomerate, approximately 200 cm wide. Up to 50 cm of fine-grained, siliceous hangingwall quartzite is exposed. The footwall is a slightly argillaceous yellow-grey quartzite. At the first pillar that was examined, the hangingwall was found to be intact without mining induced stress fractures. The reef in the pillar had fractures (4 per m) that dip at 65 ° into the gully (Figure 2-14 and Figure 2-15). There was sliding and tilting of the footwall, especially into the gully. The fracture frequency in the footwall increased from 8 fractures per m to 32 per m near the gully. The dip of the fractures also increased from 48 to 65 ° towards the gully.

![Stratigraphic column of Gold Mine B, site 1. Note light grey (siliceous) hangingwall and several discontinuous shale bands.](image-url)

Figure 2-13: Stratigraphic column of Gold Mine B, site 1. Note light grey (siliceous) hangingwall and several discontinuous shale bands.
The second pillar was characterized by an intermediate dipping (55 °) 5m wide dyke, which cut through the pillar (Figure 2-14). Within the dyke the fracture frequency dropped down to 5 fractures per m. Either side of the dyke, frequencies of 50 and 30 fractures per m were recorded. Their dip was steep – 75 ° and outwards from the pillar. The hangingwall was formed by separation along the shale-gritty quartzite contact. This is a very low cohesion surface, as well preserved ripple marks, which formed during the deposition of the sediment, were observed.
on it (Figure 2-16). The fracturing in the pillar dips in the opposite direction to the previous pillar (see Figure 2-14), suggesting that it is the distance to the nearest opening in the footwall (either strike or dip-gully) that determines the dip direction of the footwall fractures, rather than a pre-existing geological fabric.

Figure 2-16: Pillar 2 hangingwall conditions showing two orthogonal joint sets and ripple marks along hangingwall contact.

In the third pillar at Gold Mine B, Site 1, two pre-existing drill-holes into the pillar were logged using the borehole camera (Figure 2-17). These holes produced a Pillar Fracture Index (PFI) of 21 and 27. The holes were only 0.75 m and 0.57 m deep and thus cannot be said to be indicative of the true PFI of the pillar, but rather represent the maximum PFI, with the core of the pillar probably having a lower fracture density.

Figure 2-17: Borehole logs of the fractures mapped in drill-holes in Pillar 3 at Gold Mine B, Site 1.

There appeared to be outward extrusion along a 4 cm shale-band 1/3 of the way up the pillar. The bottom reef contact also had a thin 2 cm shale-layer, which allowed the slabs in between to
be rotated outwards from the pillar, and dip at 65 °. The upper 2/3 of the pillar (200 cm) had fractures dipping inwards at 70 ° (Figure 2-18).

![Figure 2-18: Cross-sectional view through Pillar 3 showing how fracture orientation and frequency is controlled by stratigraphy. Indicated dips are in degrees.](image)

Similar fracturing was noted in the other pillars. Figure 2-19 shows that the fractures on the edge of the pillars tend to bow outwards 1/3 of the way up, typically on the shale band mentioned previously. The thicker this shale band, the shallower the dip of the fractures, suggesting perhaps that more horizontal movement can be accommodated along the wider shale bedding planes.

![Figure 2-19: Length and angle of fractures noted in pillars 1 to 3 at Site 1. Length in cm is shown on the left and angle (dip) is indicated on the right.](image)

The fourth pillar had fracturing very similar to the previous pillar. A talc-coated top reef contact with slickensides was visible. The 190 cm of reef above the 20 cm finely laminated shale layer was intensely fractured (36 fractures per m), with fractures dipping inwards at 60 °. Horizontal shearing was evident within the shale layer and it had flat dipping fractures spaced less than 1 cm apart. [A similar condition was found at Platinum Mine E, where the horizontal movement...]

38
instead occurred along a weak chromitite band.] The footwall quartzite was less intensely fractured (4 fractures per m) but up to 2 cm of separation was apparent across the fractures.

The fifth pillar was undersized (5 x 4 m) and quite badly damaged (Figure 2-20). Fracturing appeared to extend 60 cm into the pillar. The top 2/3 of the pillar had a fracture frequency of 28 per m, dipping at 75° into the pillar. The lower 1/3 had a frequency of 12 per m, but these were at least 2 cm open (Figure 2-21). This represents a horizontal footwall movement of over 10 cm.

Figure 2-20: Photograph of Pillar 5, showing the intensely fractured reef, moderately fractured foot wall and essentially intact hangingwall falling out on low cohesion bedding planes.
2.2.2.2 Gold Mine B – Site 2

At this site the pillar was long and thin (20 m by 5 m). It thus does not conform to the classic crush pillar shape. Both the exposed reef and hangingwall were completely crushed (25 fractures per m), dipping at 82 and 76 ° respectively, into the pillar. The footwall had approximately half the density of fractures (12 per m), but all were open. The hangingwall above the gully was relatively intact with very little mining induced fracturing visible. At the mining face adjacent to the pillar a high frequency of fractures within the reef (72 per m), as well as movement along clay/talc coated bedding planes was observed. The fractures do not penetrate through the shale layer on the top reef contact.

More regular, in-panel pillars with dimensions of 10 x 10 m were also examined. The reef in these pillars had steep (76 °) fractures lying sub-parallel to the pillar edge and occurring at a rate of 22 per m. The footwall also had steep dipping fractures (75 °) but these were spaced 10 cm apart on average and showed at least 0.5 cm dilation on each surface. These fractures do not appear to have a preferred orientation where they were mapped in the holing. The fracture orientation and spacing is thus similar for both pillar types. Where the shale band is present in the hangingwall there appears to be less fracturing but more dilation in the footwall and the footwall heave is greater.

2.2.2.3 Gold Mine B – Site 3

Pillars at Site 3 were small and rectangular in shape, approximately 3 m long and 1 m wide. At this site, there was a conspicuous lack of shale bands within and around the reef. Figure 2-22 is a stratigraphic column of the area showing these characteristics. The reef channel was much thinner at this site than elsewhere (50 cm vs. about 200 cm). This meant that instead of seeing movement along stratigraphic layers, there was a large amount of damage to the pillars and the footwall. The hangingwall once again remained relatively intact and smooth.
Figure 2-22: Stratigraphic column of Gold Mine B, Site 3.

Figure 2-23 shows the orientation and frequency of the fractures in the first pillar mapped at Site 3. Of note is the fact that the fractures are oriented in a down-dip direction rather than towards the adjacent siding and gully. The PFI of this pillar (determined by mapping of the holing shown in Figure 2-23) was only 16. A hangingwall shale parting was very rarely present but where it occurred, even as a thin bedding-plane contact, it resulted in a smooth, relatively un-fractured hangingwall. Footwall damage appeared to be restricted to these areas with smooth hangingwall. Thus in the case of the first two pillars mapped at Site 3, the damage was mainly to the reef and then to the footwall.

Figure 2-23: Plot of fracture spacing across the holing in the pillar at Site 3 showing position of section-line.

In the third pillar, the level of fracturing was less than the other pillars in this area, due to the presence of a large gouge-filled shear. This shear, and its associated en-echelon fractures, were coated with up to 1 cm of rock-flour, indicating that a large dynamic deformation occurred to the rock mass comprising this pillar. The top reef contact was clay-coated, but no movement indicators such as slickensides were detected on it. This suggests that the shear fracture developed before the pillar was formed (once the pillar was cut, the forces required to develop the shear in solid rock would have most likely sheared this low-cohesion top reef contact first). When the pillar started taking load as a result of the mining-induced stresses, it was able to shed it by movement along the shear, rather than the development of mining-induced fractures.

At the next pillar (Pillar 4), a bedding parallel mylonite fault was evident in the hangingwall. Shearing had occurred along this surface, but it was not possible to tell if this was of geological origin or subsequently mining-induced or enhanced. However, the intensity and depth of fracturing in the pillar and into the footwall was high. The fracture frequency was 61/m, dipping at 81° away from the gully (that is down-dip along the reef plane). The fractures do not dip towards the nearest opening, as was the case at Site 1. The lack of low cohesion shale partings thus appears to play a major role in determining the orientation (but not necessarily the frequency) of the pillar fractures. The strike of the fractures in the pillar is approximately 20° up-dip of the long axis of the pillar. The pillar was also exceptionally long and narrow, being only 2.5 m wide and 8 m long. These dimensions combined with the low cohesion hangingwall contact may have caused the high frequency of fracturing. In the rock mass surrounding the
fifth pillar mapped at this site, it was noted that the hangingwall had almost no fractures, whilst the reef and the footwall are quite intensely fractured, with a fracture frequency of 48 and 20 fractures per m respectively. The reef fractures dip at 65° into the gully and the footwall fractures dip at 45° into the gully. A PFI of 10 was calculated for this pillar.

2.2.2.4 Gold Mine B – Site 4

Unfortunately it was not permissible to take a camera underground at this site. Two pillars were examined in an extensively mined stope, although adjacent raises had not been mined due to poor grade. The first pillar (a proper crush pillar) was situated in the centre of the mined out area and the second was a pillar at the edge of the workings. It was reported that the area updip of the pillar had been mined out on a higher stratigraphic elevation than the current stope. The hangingwall in this area is a siliceous grey quartzite, the reef a small pebble conglomerate with abundant pyrite stringers and the footwall is an argillaceous grey quartzite. The stratigraphy is thus very similar to that of Site 3. However the pillars are smaller and more infrequent at this site.

A single pillar (3.8 x 3.2 m) in the middle of a mined out area extending for tens of meters in all directions was mapped (Figure 2-24). The most significant observation was the large cracks developed in the footwall, particularly on the down-dip side, around the pillar. These cracks were 10 cm wide, approximately 3 m long and extended into the footwall for about 1 m. The pillar appeared to be completely crushed with a fracture frequency of 80 / m. The hangingwall was also fractured at a frequency of 20 / m, up to a smooth shale-band contact. The footwall fracture frequency was 56 / m but many of these were wide open.

![Figure 2-24: Plan view of the pillar at Gold Mine B, Site 4, showing the orientation and position of the footwall cracks. These are dilating in a down-dip direction towards the raise to the right.](image)

The second pillar examined was not properly cut, and abutted directly onto un-mined solid next to a dyke. At this pillar the reef was extensively crushed into small, 10 cm, pieces. The footwall was broken into larger pieces about 10 cm wide and 1 m long. It appears as though the footwall is dilating outwards and down-dip into the centre gully, perhaps moving along bedding planes in
the footwall. There was evidence of this movement in the centre gully. However these features are not considered typical of a crush pillar and were merely noted for comparison.

2.2.3 Gold Mine C

At this mine, there has been extensive mining. The investigation sites were deep (between 3150 and 3450 m below surface) and surrounded by mined out areas. The effects of the high stresses were clearly evident, not only on the pillars, but also within the stope in general. Intensive MIFs, shears associated with seismic events and extensive dog-earing of the monitoring holes were evident at the various sites at this mine. Panel spans were approximately 20 m with stopes 100 m apart. Strike-oriented pillars (3 x 8 m) were cut on the down-dip side of the gullies, resulting in a pillar height of between 2.4 m and 0.8 m.

2.2.3.1 Gold Mine C – Site 1

A monitoring site was established in May 2005. However, due to retrenchments at the mine, mining activities at this site stopped and the site had to be abandoned. Preliminary mapping and PFI investigations showed abundant extension fracturing of the reef within the pillar (see Figure 2-25). Large scale shears within the reef were also mapped at the site, indicating that the rock mass is being affected by seismicity (confirmed by the mine rock engineers).

![Figure 2-25: Fracture orientations at Site 1 Gold Mine C.](image-url)
The site had a relatively smooth hangingwall and fractured reef and footwall (Figure 26). Similar conditions were encountered at a second monitoring site (Mine C Site 2) established in June 2005. Figure 2-27 is a log of the fracturing within the pillar (a & b) and within the hangingwall (c). It can be seen that the fracturing in the pillar was a lot more intense than in the hangingwall, and indicated a higher stress concentration. Even though the holes were logged within minutes of their completion, large portions of the hole were already dog-eared (uncoloured portions in Figure 2-27 a and b). This indicates that the pillars are actually crushing behind the face – Figure 2-25 shows the relative positions of the mining-face and the boreholes.

Figure 2-26: General rock mass conditions at Site 1, including smooth hangingwall (left) and intensely fractured reef (centre) and footwall (right).
2.2.3.2 Gold Mine C – Site 2

At Site 2, the instrumentation was installed a few days before cutting of the pillar had commenced (Figure 2-28). It was intended to capture the data of the entire “life-cycle” of the pillar, but ground conditions deteriorated and mining was abandoned at this site. Monitoring thus only occurred weekly for a month. Logging of the extensometer and camera borehole in the face and the strike edge of the pillar showed that the PFI (before the pillar was cut) was 52 and 66 respectively. Unfortunately due to subsequent blockages near the collar of the borehole it was not possible to compare the time-dependant changes in the PFI. Closure-ride stations were installed downdip and along strike of the planned pillar position (Figure 2-28). These were labelled Station 1 and 2 respectively. The closure-ride measurements for the month that monitoring was possible, indicated that the daily closure was between 0.55 and 1.2 mm per day, with no evidence of this rate decreasing (see

Figure 2-29 and
Figure 2-29). The total closure over the monitoring period was 16.5 and 36 mm. During the same period, pillar dilations of 31 mm and 20 mm were measured on the extensometers installed in the pillar boreholes adjacent to the closure-ride stations.

*Figure 2-28: Instrumentation and geology of Gold Mine C, Site 2.*
Figure 2-29: Installation of closure-ride station bolts at Site 2, Gold Mine C. Note the extremely steep dip.

Figure 2-30: Closure measured at Gold Mine C, Site 2, Station 1.

Figure 2-31: Closure measured at Gold Mine C, Site 2, Station 2.
2.2.4 Platinum Mine A

A single site was investigated and geotechnical instrumentation was installed under the auspices of SIM040207. At this site mining was occurring on the Merensky Reef at a depth of approximately 1000 m below surface; with dip oriented 3.5 x 2 m crush pillars being cut.

The footwall was a fine-grained grey norite. In this area, the Merensky Reef consists of a coarse-grained feldspathic pyroxenite overlain by a medium- to coarse-grained norite, often with euhedral, brown, pyroxene grains. Two major geological sets of discontinuities were present, indicated by solid lines on Figure 2-32, whilst the mining-induced fractures are indicated with dotted lines. The position of the borehole camera survey holes is also indicated on this plan.

Table 2-4 summarizes the characteristics of these and the mapping of the mining-induced discontinuities.

![Figure 2-32: Detailed plan showing the orientation and spacing of the mapped discontinuity sets at Platinum Mine A. Borehole camera holes are in the lower of the three pillars illustrated.](image_url)
Table 2-4: Characteristics of discontinuities mapped at Platinum Mine A.

<table>
<thead>
<tr>
<th>Set</th>
<th>Dip</th>
<th>Strike</th>
<th>Position</th>
<th>Spacing</th>
<th>Infill</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>55</td>
<td>300</td>
<td>FW, reef, HW</td>
<td>2 m</td>
<td>Minor talc &amp; chlorite</td>
</tr>
<tr>
<td>2</td>
<td>68</td>
<td>190</td>
<td>FW, reef, HW</td>
<td>7 m</td>
<td>Clean</td>
</tr>
<tr>
<td>3</td>
<td>30</td>
<td>110</td>
<td>HW</td>
<td>0.25 m</td>
<td>Clean</td>
</tr>
</tbody>
</table>

Set 3 discontinuities formed during the blasting of the down-dip face. These discontinuities terminate against the pre-existing geological weakness of Sets 1 and 2. Set 1 discontinuities were common within the footwall norite of the gully. This is because the gully is running orthogonal to the strike of these discontinuities and parallel to Set 2 and hence intersects more of Set 1. In general the footwall norite is slightly more fractured, despite its presumably higher UCS. It is likely that the higher plagioclase content of this norite makes it more brittle, causing it to fail. The crush pillars have a nominal dimension of 3.5 x 2 m but vary greatly from this, particularly in length, being up to 7.5 m long. The longer pillars do not appear to be crushing completely. The formation of the crush pillars changes the orientation of the low angle MIFs to pillar-edge parallel, up to 2 m from the pillar edge.

Closure meters and closure-ride stations were installed in the panel to measure the movement of the rock mass for SIM040207. With regular blasting, the closure rate at a distance of 14 to 20 m from the face was found to typically be 2 mm per m. Malan (2005) notes that this value should be treated with caution, however, as the majority of the closure at this distance to face appears to be time-dependent. The rate of steady-state closure can vary from as high as 2.2 mm per day after blasting to as little as 0.14 mm/day. On the updip side instrumentation site, an oversized crush pillar (7.5 m long) failed violently and ejected material into the gully (Figure 2-33). The discontinuities of Set 1 were clearly visible across the pillar, apart from its updip edge. In addition, the ejected pillar material is visible in front of these features. This suggests that during the formation of the pillar much of the stress that should have resulted in the fracturing of the reef was actually accommodated along the discontinuities of Set 2. The pillar was thus unable to start crushing and yielding. As the load increased with further mining, these features were no longer able to move and yield, resulting in a stress build-up, which could not be relieved through movement of the fractured reef zone.
Borehole camera examination of the holes drilled into a crush pillar that was being formed (see Figure 2-32 and especially Figure 2-34) indicated that fractures of Set 2 were present to the centre of the pillar. However, as discussed below, the frequency of these discontinuities was relatively low. Hole 1 and 4 were 4.3 and 0.5 m respectively from the mining face on the day they were investigated and both showed fracturing to the end of the hole. Hole 2 was drilled immediately before the installation of the borehole camera, but had already collapsed 15 only cm into the hole. This suggests a high level of fracturing and movement in the rock mass. The PFI of the holes varied from 56 to 0 to 67 to 20 for holes 1 to 4 respectively. This high degree of variation is because holes 1 and 3 (PFI = 56 and 67) were drilled to the centre of the pillar, whereas hole 4 was drilled at an angle and as such remained in the outer fractured portion of the pillar. Hole 2 (PFI = 0) was probably too short to be representative of the pillar fracturing. These numbers indicate that the pillar was behaving more like a yield than a crush pillar, as yield pillars have a PFI > 45 (Grodner and Canbulat, 2005). This is probably because even though the pillar had been designed as a crush pillar, it was actually oversized by about 45%.
2.2.5 Platinum Mine B

At Platinum Mine B, in-stope pillars are used extensively. Although these are designed and usually cut as yield pillars, their size sometimes varies to that more typical of a crush pillar. Information on yield pillars was also useful for the understanding of the variation in rock mass behaviour with different pillar types. From the data gathered at the various sites, it does appear as though there is a continuum in rock mass behaviour (including fracturing, closure and pillar dilation) between these two pillar types. Of particular interest is that pillars that initially had a PFI of a yield pillar, dropped to a crush pillar PFI after several weeks. This indicates that these pillars may have the same ultimate strength, but reach this at different rates. It also suggests that a significant amount of the fracturing of crush pillars does not occur ahead of the face.

2.2.5.1 Platinum Mine B - Site 1

Several small potholes were indicated on the stope plan of the area (Figure 2-35) and it was also observed that the bottom reef contact is highly undulating with rolls of up to 20 cm (Figure 2-36) which is often associated with the onset of potholing.

Figure 2-34: Sketch plan and section showing the position of the borehole camera holes.
Figure 2-35: **Stope-plan of Platinum Mine B, Site 1, located in the central panel.**

![Stope-plan of Platinum Mine B, Site 1](image)

Figure 2-36: **Photograph showing the undulating bottom reef contact.**

At this site, information was collected of rock and discontinuity characteristics. Figure 2-37 is a combined geological log of sidewall mapping and mapping with a borehole camera. From this it can be seen that the hangingwall consists of feldspathic pyroxenite. The Merensky Reef is approximately 20 cm of coarse-grained pyroxenite with a well developed bottom chromitite. The footwall grades rapidly from a spotted anorthosite into a leuconorite. These more brittle, feldspathic host-rocks may be the major controlling factor for rock mass behaviour in this area, which shows limited closure and is relatively active seismically (pers. comm., Miovsky 2005).
Figure 2-37: Stratigraphic column of the rock-types observed in side-wall and borehole mapping. Numbers indicate thickness of rock-types in cm.

The Merensky Reef dips at 12 °, but the undulating bottom reef contact may prevent any layer parallel shearing. A single geological joint-set was observed, spaced 1.5m apart, constituting a stiff rock mass (Figure 2-38). Within the panel, the fracturing was dominated by low angle (15 °) mining-induced fracturing (MIF) in the hangingwall (Figure 2-39). The face and footwall fracturing is steeper (75 °). The condition of gully hangingwall is controlled by steep dipping strike-parallel fractures. These disappear rapidly and were not observed in the hangingwall on the up-dip side of the gully.
The face-parallel discontinuities have a higher frequency than those associated with the pillars along the gully, and are restricted to the reef and the immediate hangingwall rock, before being deflected to form the low angle hangingwall discontinuities. Two boreholes were geotechnically logged – these are shown on Figure 2-38 as “observation holes”. The first hole was drilled at 75º into the hangingwall for 1.3 m. Surprisingly, no discontinuities of any kind were observed in this hole, giving it a PFI of 100. A second hole was drilled flat into the pillar, from the siding. The average spacing of the discontinuities is 0.17 m, with a PFI of 89. This high PFI and the characteristics of the few discontinuities that were observed in this hole, suggests that the pillar was not taking much load. This may be due to the large lag of the panel below it. The closure was consistent across the panel for the month that it was monitored, varying between 14 and 17 mm. During the same period, pillar dilation of 25 mm was measured on the extensometers in the observation hole in the pillar.
2.2.5.2 Platinum Mine B - Site 2

At Site 2, mining is also occurring on the Merensky Reef but in a breast configuration with yielding pillars. There are two geological joint sets with dips of 55 and 85° at this site (Figure 2-40). The calcite coated geological discontinuities of the second set occur in a distinct zone, suggesting that they are part of a fault, rather than a regular joint-set. In a very similar manner to Platinum Mine B, Site 1, a shallow dipping set of mining-induced fractures are oriented towards the face (Figure 2-41). An additional set of steep dipping mining induced fractures are developed on the updip side of the pillar in the gully, possibly due to the lack of low cohesion stratigraphic partings in the immediate hanging or footwall.

The pillars have a fractured outer edge and intact core – Figure 2-42 is a photo mosaic of a holing through the pillar showing this. An overall PFI of 79 was calculated for this pillar, but the outer meter only has a PFI of 41. Thus it is important to take measurements into the centre of the pillar to determine the actual PFI. A PFI of 79 correlates well with the initial PFIs determined for other partially formed crush pillars.

Figure 2-40: Sketch-plan of the orientation of the discontinuities at Platinum Site 2.
Figure 2-41: Photograph of the low angle MIFs dipping towards the face (machine operator's right).
2.2.5.3 Platinum Mine B - Site 3

At this site, several reasonably sized (panel scale) potholes were present and it was also observed that the bottom Merensky Reef contact was highly undulating - indicative of even more potholing. The hangingwall consisted of at least 1.75 m of fine grained pyroxenite. The reef is approximately 20 cm thick, and the footwall is a leuconorite. A single geological joint-set was observed, spaced 1.5 m apart. As at Sites 1 and 2 of Platinum Mine B, low angle (15 °) MIFs occur in the hangingwall. The face and footwall fracturing is steeper (75 °). The gully hangingwall, next to the pillars is dominated by steep dipping strike-parallel fractures (Figure 2-43). Extensometers were installed in one of these holes and the other was monitored weekly using a borehole camera. PFI values were determined for this pillar for each of these times. More detail on the methodology for the calculation of the PFI is given in Section 2.4 and as such it is not discussed here. There is a noticeable decrease in the PFI with time. This time-depndant decay of the PFI is more sensitive to changes in fracturing than simply taking the average spacing of the fractures, and can also be correlated with specific measurements from the other instrumentation at the site. Figure 2-44 shows the time-dependent change in fracturing in the borehole – this hole is labelled BHC in Figure 2-43.
At Site 3, in addition to the normal geological and geotechnical characterization of the site, several rock mass monitoring systems were installed. These were three closure-ride stations, one clockwork closure meter and two holes drilled into the pillar (Figure 2-43). The total closure was 23 mm for the 5 week monitoring period (Figure 2-45).

**Figure 2-44: Time dependant fracture changes in borehole.**

**Figure 2-45: Closure profile of Site 2, Platinum Mine B.**
The graphs below (Figure 2-46 and Figure 2-48) are of dilation of the pillar and movement of the skin of the pillar into the gully. The decrease in length of the extensometers installed to various depths in the pillar (Figure 2-46) shows that the pillar was dilating, but to varying degrees at different depths. These extensometers indicate a maximum dilation of 54 mm (Figure 2-46) which agrees well with the collar movement of 46 mm, as indicated by Figure 2-47. Pillar dilation is roughly twice the closure (approximately 50 mm vs. 23 mm). Figure 2-48 shows the PFI as mining progressed past to form the pillars at Sites 1 and 3. The PFI shows a decrease in rate of change with time, as mining moves away from the pillar. The dilation of the pillar appears to be controlled by the amount of fractures, as the change in dilation correlates reasonably with the change in the PFI (Figure 2-47 and Figure 2-48).

**Figure 2-46:** Extensometer measurements Platinum Mine B, Site 3. The key indicates anchor distance (cm) from the end of the hole (approx. centre of the pillar).
Figure 2-47: Pillar dilation at Site 3, hole collar relative to a h/w peg above the gully.

Figure 2-48: PFI changes with time of the pillars at Site 1 and Site 3.

Rock samples were taken from Platinum Mine B, Sites 1 and 3 for laboratory testing. The results are shown in Table 2-5. The samples were taken from these sites, as the most detailed underground measurements for the longest period of time were available from these sites. This
information was intended for use in the detailed numerical modeling of pillar behaviour described in Section 3 of this report.

Table 2-5: Intact rock characteristics at Platinum Mine B, Sites 1 and 3.

<table>
<thead>
<tr>
<th>Position</th>
<th>Rock type</th>
<th>UCS (MPa)</th>
<th>Young's Modulus (GPa)</th>
<th>Poisson’s Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hangingwall</td>
<td>Fine-grained pyroxenite</td>
<td>98</td>
<td>34</td>
<td>0.35</td>
</tr>
<tr>
<td>Reef</td>
<td>Feldspathic pyroxenite</td>
<td>123</td>
<td>47</td>
<td>0.34</td>
</tr>
<tr>
<td>Footwall</td>
<td>Leuconorite</td>
<td>133</td>
<td>76</td>
<td>0.42</td>
</tr>
</tbody>
</table>

2.2.5.4 Platinum Mine B - Site 4

A fourth instrumentation site was established one level deeper than Site 1. However, due to logistical and funding issues this site was only monitored for two weeks and hence the pillar was not completely formed. The hangingwall at this site was a fine grained feldspathic pyroxenite that overlies a thin (10 cm) Merensky Reef. The footwall of the Merensky Reef is feldspathic consisting of a mottled anorthosite that overlies a spotted anorthosite grading into a leuconorite (Figure 2-49: Stratigraphic section of the strata making up the pillar at Platinum Mine B, Site 4.). Even though the reef is approximately half as thick as at the slightly shallower Site 3, the overall stratigraphy is similar, with a more pyroxenitic reef and hangingwall overlying a feldspathic footwall. Structurally, the rock mass is less disturbed, with only a single set of steep dipping geological discontinuities being present. These discontinuities were almost perpendicular to the long axis of the pillar, and hence were not observed in the borehole camera hole (BHC1 in Figure 2-50). Figure 2-50 also shows that there are two sets of MIFs, namely a shallow dipping, face parallel (20 °) set that is present in the hangingwall and a steep dipping, sidewall parallel set. These sets of discontinuities are essentially the same as what was encountered at the other sites on this mine.
Figure 2-49: Stratigraphic section of the strata making up the pillar at Platinum Mine B, Site 4.

A PFI of 69 and 73 was measured during the installation of the instrumentation at this site. These average out at 71, which is very similar to the initial PFI measured at Site 1 (see Figure 2-48). The PFI then dropped from 71 to 57; it thus appears as though the change in depth of approximately 150 m did not affect the PFI significantly (as shown by Figure 2-51). The rate of change of PFI may be an indicator of how the pillars are behaving – see Section 2.4 for more details.

For the brief period that this site was monitored a closure rate of 1.4 mm per day was determined, giving a total of 7 mm closure. Over the same time, a dilation of 5.4 mm at the future site of the pillar was measured.
Figure 2-50: Planned monitoring Site 4, Platinum Mine B.
2.2.6 Platinum Mine C

Under the auspices of another project it was possible to visit a site at Platinum Mine C. Updip mining of the UG2 was occurring at a depth of approximately 1000m below surface, utilizing in-stope crush pillars. The rock mass conditions were relatively poor, with an extremely blocky hangingwall. This blocky hangingwall is caused in part by the abundant pegmatoidal veins cutting through the stope. In addition, the reef rolls into the footwall on the face. These geological conditions are normally indicative of a nearby pothole. Six different sets of discontinuities are present in this area. The position and orientation of the mapped discontinuities is shown in Figure 2-52. Discontinuity sets 1 to 3 are of geological origin and sets 4 to 6 are probably as a result of a combination of geological and mining induced stresses. The latter three sets have a strike orientation very similar to that of discontinuity set 3, but with varying dips due to the influence of mining activities.

Figure 2-51: PFI changes with time at Site 4, Platinum Mine B, compared with the longer period of data from Sites 1 and 3.
The reef dips at approximately 14°. The UG2 chromitite was approximately 95 cm thick and is overlain by a fine grained melanorite. Despite the very blocky hangingwall conditions none of the overlying chromitite stringers were visible. The UG2 is developed on the top of a coarse grained leuconorite which grades downward into a finer grained leuconorite. The leuconorite is approximately 10 cm thick, but in places it appears to have been “eroded” by the chromitite. Underneath this, a pegmatoidal norite is developed. This coarse grained rock appears to be easily fractured, with abundant slabs forming along the sidewall of the pillar. The pegmatoidal norite is about 0.5 m thick and overlies a fine grained leuconorite.
Two of the holings between the crush pillars were mapped (e.g. section AB in Figure 2-52). The pillars were highly crushed, with a PFI of 46 and 25 being calculated for the two sections investigated. These low PFI values are probably due less to the highly fractured nature of this site and more to fact that the area was mined out two months before the holings were mapped. Considering the age of the pillar, the PFI values are similar to those measured at Platinum Mine B, ten weeks after the pillar had been cut. This is despite the fact that these pillars were formed on different reef horizons.

### 2.2.7 Platinum Mine D

Detailed underground mapping of geotechnical and mining conditions was carried out in areas where poor ground conditions were associated with crush pillars. Several (39) pillars at 19 sites on Platinum Mine D were characterised. The relationships between the various parameters were examined to determine which of characteristics were critical in determining pillar and rock-mass behaviour.

Poorer ground conditions in areas dominated by mafic rocks may be as a result of the lower cohesion on the discontinuities present (due to the greater degree of weathering of rocks of this composition). Footwall (pillar sidewall) failure tended to occur later in the mining cycle, after the gullies had been cleaned to their full depth. This exposed a brittle fractured, anorthositic footwall. The damage to the gully sidewall was caused by heavily loaded but uncrushed pillars, exacerbated by the lack of sidings. If the pillars are too strong, footwall heave will cause slabbing into empty gullies. The presence of empty gullies was found to be critical for this type of failure, indicating that sidewall slabbing was dependant more on mining phase than geotechnical condition. The applied stresses do not appear to be a controlling factor, as these were similar for both failed and intact sidewalls. If the rock mass is relatively strong, with a competent hangingwall and limited joints, the stress caused by oversized, uncrushed pillars and larger spans with larger pillar w/h ratios cannot be relieved by a weaker part of the system failing in a gradual manner. As such, stress builds up and the pillars could burst. This was observed where there were one or no joint sets and both the hanging and footwall consisted of stronger felsic rocks (either anorthosite or leucorinite). Generally it appears that bigger pillar sizes and spans result in worse conditions and hence the cutting of bigger pillars in problematic areas tends to worsen the problems.

Although no clear correlation was found between pillar behaviour and any single characteristic, a combination of geological characteristics and resultant geotechnical of the rock mass surrounding the mining led to fairly consistent responses. This highlights the fact that the pillars are part of a system that also includes the hanging and footwall. If failure occurs, it is focused on the weakest part of the system. Thus both hangingwall and footwall failure can be as a result of poorly designed or mined pillars, as well as failure of the pillar itself (}
Figure 2-53: (A) Hangingwall, (B) footwall and (C) violent pillar failure of poorly designed or mined pillars on the same reef at similar depths on Platinum Mine C.

2.2.8 Platinum Mine E.

Although this work was not performed as part of SIM 040302, certain of its findings are relevant to crush pillar behaviour and pillar behaviour in general. It thus serves as a useful comparison between the geological and geotechnical conditions present at the crush pillar sites and the subsequent rock mass response. Monitoring of pillar conditions at Platinum Mine E was
conducted to verify the design of the bord and pillar layout. At this site, a 1.8 m cut is taken on the Merensky Reef. The immediate hangingwall is a pyroxenite overlain by norite and spotted anorthosite to a thickness of between 4 and 20 m. The immediate footwall is a leuconorite. The mining depth is approximately 850 m below surface. The panels are 12 m wide and are separated by pillars, which are nominally 6 m wide on strike and 4 m wide on dip with 6 m wide holings. These panels were situated between large stability pillars to the west and an abutment to the east, providing a maximum span of approximately 60 m. A number of boreholes were drilled to a depth of approximately 10 m into the hangingwall and footwall to observe initial partings and fractures and for the installation of extensometers. Although the wide ranges of pillar dimensions actually cut were undesirable from the mine’s point of view, the site provided an excellent opportunity to quantify the effect of pillar dimensions on their condition.

Pillar condition was quantified by measuring the depth and intensity of fracturing and from this information the PFI was calculated. A contour map of the PFI values across the area was generated (Figure 2-54). Of interest is the fact that even at the early stage of monitoring, the PFIs of the smaller grid pillars was significantly lower than the strike barrier pillars and the values measured in the abutment (Figure 2-54). Grodner and Canbulat (2005) concluded that crush pillars have a PFI less than 45, yield pillars have a PFI between 45 and 90 and solid pillars have a PFI greater than 90. It appears as though the grid pillars are acting as either crush or yield pillars, as their PFI is less than 90 and occasionally drops to below 30. The strike barrier pillars are remaining as solid pillars, with a PFI of greater than 90.

Figure 2-54: Contour plot of the PFI - values less than 90 (below the red contours) indicate that the pillars are yielding. The blue contours suggest that some of the grid pillars are actually acting like crush pillars.

During a subsequent follow-up visit to this site, abundant pillar fracturing and dilation was noted on the smaller grid pillars. Similar to Gold Mine B, there was dilation outwards along the weak
layer within the pillar. Fracture orientations and frequencies were similar, with the only difference being that at this site, the weak layer was the basal chromitite of the Merensky Reef, whereas at Gold Mine B, it was a shale band.

2.2.9 Platinum Mine F

Although this mine is utilizing stable and not crush pillars, it was decided to do observations on the pillar fracturing to obtain a relative measurement of the rock mass conditions, especially the PFI in stable pillars (Figure 2-55). The PFI values for this mine were all above 90, which is significantly different from those obtained for crush pillars. This finding indicates that the PFI can successfully discriminate between stable and crush pillars.

Figure 2-55: Plan view of observation hole positions in the stable pillar.
2.3 Seismic Monitoring

Several attempts were made to install seismic instrumentation at Gold Mine A. Installation of the seismic monitoring system was very difficult, despite assistance from the mine in terms of access to their mine-seismic network. The placing of cabling into the stope to facilitate pillar monitoring was physically demanding and awkward. In addition several problems were experienced during the mounting of the geophones to the rock surface. This is because the ideal position of the sensors for location of seismic events within the pillar is against the reef. It was initially attempted to install instrumentation at the face (Figure 2-56a) whilst the pillar was being cut. Slabbing of the face and fall-out of the tuffaceous hangingwall made this impractical as the coupling and hence location of events would be poor. As a compromise, two accelerometers were installed on the hangingwall next to the pillar (Figure 2-56b) and monitored for a week, before being moved closer to the mining face for the second week.

Figure 2-56: (a) Protective metal tubing to prevent blast-damage to seismic instrumentation at the face, (b) Accelerometers installed on hanging wall

The results of the seismic monitoring can be summarised as follows:

Two weeks of valid seismic data (615 events) were eventually recorded.

Source parameters of the recorded events are in accordance with physical considerations of the monitoring site.
Microseismicity associated with pillar fracturing processes was observed. Distinct seismicity patterns in space and clear trends over time were observed. Most significantly, no direct pillar response to large external events was seen in the seismic monitoring.

The two-station network (i.e. the hangingwall accelerometers) only allowed six components to be recorded at all times. This meant that the directions for location were sometimes ambiguous, but this was offset by the fact that there were low noise levels and thus a high sampling frequency was used. The instrument configuration was sensitive to indirect ray paths, including reflected, surface and head waves (Figure 2-57). The multiple arrivals on only two sensors with short P-S separation and very short arrival time differences (due to the close proximity) meant that a limited database of reliable local events was recorded. These issues negatively affected the local event location accuracies.

![Cross-section showing the sensor positions, variable seismic sources and ray paths to the accelerometers. Note the slabbing of the pillar which prevented mounting of the sensors to the face.](image)

The decision to use ISSI equipment resulted in a compromise in terms of spatial and temporal resolution. Due to hardware limitations it became impossible to locate events within two metres of the sensors and to distinguish between events that occurred within 0.5m of each other, as was originally intended. It took several months to resolve technical faults and make the set-up work. One of the sensors had to be replaced, but failed again after two weeks. The budget for seismic equipment was insufficient to import more appropriate Canadian equipment from ESG with the necessary specifications. There was also no suitable equipment available from previous SIMRAC projects that could have been used. Instead, the following ISSI equipment was used at its upper performance limit:
Seismometer: ISSI QS with 24kHz per channel and dynamic range >110dB, linked to mine seismic network.

Sensors: Two tri-axial accelerometers 3A10k (ISSI) with usable frequency range up to 10kHz; configured as a local sub-network to associate two-station events or to associate into global mine events.

For data transfer, the mine allowed the temporary use of a mine QS box for the duration of the project. This box was modified by ISSI to process accelerometer data. On the local pillar monitoring network, 615 events were recorded. Of these 372 events (60%) had valid 3D location, and only 45% had P- and S-energy signatures. During the same time period the mine’s network recorded 1615 events. The characteristics of the largest six of these events are shown in Table 2-6. Figure 2-58 and Figure 2-59 show examples of valid seismic events that were used in the analysis.

Table 2-6: Characteristics of the six largest events recorded over the pillar monitoring period.

<table>
<thead>
<tr>
<th>Distance</th>
<th>Moment</th>
<th>Energy</th>
<th>Magnitude</th>
<th>Pillar Events</th>
<th>Pillar energy</th>
</tr>
</thead>
<tbody>
<tr>
<td>387</td>
<td>3.2E+09</td>
<td>9.4E+02</td>
<td>0.6</td>
<td>None</td>
<td></td>
</tr>
<tr>
<td>202</td>
<td>1.9E+10</td>
<td>4.4E+03</td>
<td>0.9</td>
<td>6</td>
<td>7.9J</td>
</tr>
<tr>
<td>637</td>
<td>3.6E+10</td>
<td>6.9E+03</td>
<td>1.0</td>
<td>5</td>
<td>0.6J</td>
</tr>
<tr>
<td>582</td>
<td>4.6E+09</td>
<td>2.0E+03</td>
<td>0.7</td>
<td>5</td>
<td>4.1J</td>
</tr>
<tr>
<td>380</td>
<td>2.1E+09</td>
<td>2.4E+03</td>
<td>0.6</td>
<td>None</td>
<td></td>
</tr>
<tr>
<td>378</td>
<td>6.0E+09</td>
<td>2.4E+03</td>
<td>0.7</td>
<td>None</td>
<td></td>
</tr>
</tbody>
</table>

Figure 2-58: Seismic event recorded during pillar monitoring at Gold Mine A.
Location accuracy was limited by array configuration, poor alignment of sensors with the survey grid and complex wave ray paths. This meant that the location error was probably in the region of 5 to 15 m with a mix of random and systematic errors. Where events were located too far, attenuation was over-compensated for and source parameters were too large (by a factor of roughly 10). In many cases, the seismic energy was too small and seismic moment too large due to a lack of P-wave quantification for good locations. The event frequency is a combination of actual seismic activity, equipment performance and wave form complexity, and many triggers were rejected due to low quality waveforms that were recorded as a result of these problems. If an energy-moment plot of the events recorded is created it is possible to identify distinct clusters of events. Figure 2-60 is such a plot and the events with the lowest energy and moment (indicated in dark blue) are interpreted to represent the pillar fracturing events. The light blue and yellow clusters represent seismic events associated with nearby face fracturing and more distant events respectively.
It was found that there was a shift in the locations with time during the monitoring period. From Figure 2-61 it can be seen that there is a progressive movement of the events towards the North, away from the southerly pillar. The east-west scatter away from the pillar (indicated in grey in the centre of the figure) is due to the less than ideal location of the sensors in the hanging wall. In this picture it can also be seen that two weeks after the pillar had finally been formed there was no longer seismicity in it. This means that the fracturing had ceased. This agrees with the PFI data measured at Platinum Mine B, which indicates that the rate of change of the PFI decreases once the adjacent panel is mined past, also indicating a cessation of fracturing.

**Figure 2-61: Location of seismic events associated with a pillar at Gold Mine A.**

Thus despite the major technical and physical difficulties associated with seismic monitoring of a crush pillar, it has been possible to show firstly when the fracturing of the pillar occurs relative to the face position and secondly that crush pillars scarcely respond to larger distal events.
2.4 Pillar Fracture Index

During the monitoring and collection of a wide variety of underground geotechnical and rock mass response characteristics, the concept of the Pillar Fracture Index was developed. The Pillar Fracture Index (PFI) is simply an adaptation of the Rock Quality Designation (RQD) developed by Deere et al (1967). The difference is, where-as the RQD is measured from drill core, the PFI is obtained by borehole camera observations into a drill hole. The PFI is very similar to RQD measured from core from the same hole, perhaps only slightly higher if the core has been mistreated and broken during transport. It may thus be considered to be a truer reflection of the rock mass conditions than the traditional RQD measurements. It has the additional advantage that repeated measurements can be taken of the same hole to see what the time dependent changes are. When borehole breakout or “dog-earing” is encountered, that section of the hole is treated as fractured ground, even though it may not be possible to recognize individual fractures. This appears to conflict with the common belief that dog-earing is associated with high stress and thus indicates that the rock could not have fractured. It may thus be considered that it is inappropriate to treat the length of borehole breakout as fractured ground. However, it is suggested that as a measure of the conditions in a pillar, the classification of “dog-earing” as fractured ground might be appropriate. This is because when the rock is subjected to high stresses in a particular direction it can either form “dog-earing” perpendicular to, or develop fractures parallel to, the maximum stress direction. The nature of the rock mass response (either “dog-earing” or fracturing) is controlled by the confinement and not by the maximum stress level. As such, both fractures and dog-earing are indicators of the level of stress. These features, especially the borehole breakout and closely spaced fractures are often not discernable in core, making the PFI more representative of the stress conditions present. The Pillar Fracture Index (PFI) is defined as:

\[ PFI = \frac{\sum (\text{lengths of hole with fractures > 10cm apart})}{\text{Total length of hole}} \times 100 \]

It is expressed as a percentage and varies from about 20 for highly fractured pillars, to 100 for intact pillars. The PFI takes into account all the possible factors that impact on pillar behaviour from geology through to mining, as it is merely a measure of the rock mass response and not an attempt to quantify individual components. This allows clear comparisons of different sites by quantifying the fracture pattern within the pillar and representing it as a single number.

The PFI is obtained by the logging of a specially drilled borehole, or even a conventional (and very cheap) production hole, drilled into the centre of the pillar. It is assumed that the mining induced fracturing into an in-stope pillar will be approximately symmetrical about the centre axis of the pillar. Detailed mappings of holings through pillars have shown that this is indeed the case. These mappings were also used to determine the PFI. Boreholes were logged using a borehole camera such as the Flexi-Cam. During the geotechnical logging of the borehole, a tape-measure was attached to the head of the camera, and when a feature of interest was encountered; its depth was recorded by reading off the amount of tape that has unwound into the hole.
Stable (non-yield) pillars were found to have a PFI of greater than 90 (at an average of 97). Yield pillars ranged from 45 to 90, depending on their age, averaging at 58. Mature crush pillars had a PFI range of 5 to 43 with an average of 32. There is overlap between the crush and yield pillar categories, particularly with older pillars. Other potential uses of the PFI are briefly described below, but more work is necessary to properly understand these relationships to allow the PFI to become an even more useful tool.

2.4.1 Using the PFI to define the stress-strain curve of pillars

With time the PFI of a pillar will degrade downwards. Thus the PFI can also perhaps be considered to indicate where the pillar is in terms of a plot of its stress-strain curve – this is shown in Figure 2-62. Further work is however necessary to quantify the stress-strain relationship to the PFI, and a start along these lines is discussed in Section 3 of this report.

\[\text{Stress} \quad \text{High PFI} \quad \text{Moderate PFI} \quad \text{Low PFI} \quad \text{Strain}\]

*Figure 2-62: Suggested correlation between PFI and the stress-strain behaviour of a crush pillar.*

2.4.2 Time / distance dependant changes of the PFI

As mentioned previously, there is a time-dependent fall-off of the rate of PFI change – that is, the PFI first decreases rapidly, but then more and more slowly as the pillar’s age, and distance from face, increases. Although it appears as though the pillars studied will eventually decay to very similar PFIs at distances greater than 60 m from the face (Figure 2-63), there is a difference in the rate at which this occurs. The relationship between this rate of change and geotechnical areas is not clear at this stage due to the limited data. There is an obvious separation between the behaviour of yield and crush pillars, especially early in their life-cycle.
(Figure 2-63). There is also an indication that the rate of change of the PFI might be related to the rate of change of closure in different geotechnical areas. This suggests that there is indeed a hangingwall-pillar-footwall system of rock mass behaviour. Once again this relationship can only be confirmed with more data. There is however great potential if such a relationship can be defined, as it could allow the quantification of pillar behaviour based on closure. This would allow the selection of appropriate pillar types for different reefs, as Malan (2005) has quantified the closure rates for many different geotechnical areas.

**Figure 2-63: Graph of rate of change of PFI for different geotechnical areas.**

In the following section (Section 2.5.1), an attempt is made to correlate the PFI with pillar load and percentage extraction in the panel. This not a straight-line relationship, but rather a logarithmic decay of the PFI as load and percentage extraction increases. It is interesting to note that this is similar to the rate of change of PFI shown in Figure 2-63. This is because as the distance to the face increases, both the percentage extraction and the resultant load on a pillar tend to increase. However, Figure 2-63 also shows how the rate varies in different geotechnical areas. The relationship between geotechnical areas and the pillar loads with different percentage extractions is beyond the scope of the data collected to date.
2.5 Summary of Underground Monitoring

Extensive underground observations and monitoring of crush pillars has defined the failure characteristics and time-dependent changes of the rock mass of pillars and surrounding strata for a wide variety of mining environments. The mining and rock-engineering characteristics described in Section 2.5.1 are measures of the physical conditions present, whereas Section 2.5.2 describes the mechanisms of rock mass deformation. The PFI is a quantification of the rock mass deformation and can be related to the mechanisms of this. Although certain trends and uniform behaviours can be recognised by separating the conditions and the response; a pillar can only be fully described if both are considered.

2.5.1 Summary of Mining Characteristics

Table 2-7: Mining characteristics of underground sites. summarises the mining characteristics of the different sites. This information was used to derive the rock engineering parameters in

\[
\text{Tributary area} = \frac{(\text{Pillar length} + \text{Holing width}) \times (\text{Pillar width} + \text{Panel span})}{\text{Pillar length} \times \text{Pillar width}}
\]

It is expressed in square meters and when the depth of mining and rock density is taken into account, an estimate of the pillar load can be determined. This is not the actual load, as the pillar should have crushed and shed (at least) some of its load. The percentage in-panel extraction (i.e. the total panel area less the pillar area) and other values for the different sites are shown in

Table 2-8: Rock mechanical characteristics of the different monitoring sites. shown graphically in Figure 2-63 to Figure 2-65.
## Table 2-7: Mining characteristics of underground sites.

<table>
<thead>
<tr>
<th>Mine</th>
<th>Site No.</th>
<th>Depth</th>
<th>Reef</th>
<th>Panel span</th>
<th>Stope span</th>
<th>Pillar length</th>
<th>Pillar width</th>
<th>Holing width</th>
<th>Stope width</th>
<th>Mining method</th>
<th>Pillar type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gold A</td>
<td>1</td>
<td>1620</td>
<td>VCR</td>
<td>20</td>
<td>120</td>
<td>3</td>
<td>2</td>
<td>15</td>
<td>2.0</td>
<td>Breast</td>
<td>In panel</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>2250</td>
<td>VCR</td>
<td>20</td>
<td>120</td>
<td>3</td>
<td>3</td>
<td>15</td>
<td>2.5</td>
<td>Breast</td>
<td>In panel</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>1850</td>
<td>VCR</td>
<td>20</td>
<td>120</td>
<td>4</td>
<td>3</td>
<td>20</td>
<td>1.8</td>
<td>Breast</td>
<td>In panel</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>2050</td>
<td>VCR</td>
<td>20</td>
<td>120</td>
<td>4</td>
<td>2.5</td>
<td>18</td>
<td>1.8</td>
<td>Breast</td>
<td>In panel</td>
</tr>
<tr>
<td>Gold B</td>
<td>1</td>
<td>950</td>
<td>Beatrix</td>
<td>25</td>
<td>100</td>
<td>10</td>
<td>10</td>
<td>25</td>
<td>2.5</td>
<td>Breast</td>
<td>In panel</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>940</td>
<td>Beatrix</td>
<td>25</td>
<td>60</td>
<td>20</td>
<td>5</td>
<td>8</td>
<td>2.0</td>
<td>Breast</td>
<td>Strike</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>955</td>
<td>Beatrix</td>
<td>22</td>
<td>50</td>
<td>3</td>
<td>1</td>
<td>25</td>
<td>2.0</td>
<td>Breast</td>
<td>In panel</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>1030</td>
<td>Beatrix</td>
<td>25</td>
<td>40</td>
<td>3.8</td>
<td>3.2</td>
<td>29</td>
<td>2.0</td>
<td>Scattered</td>
<td>In panel</td>
</tr>
<tr>
<td>Gold C</td>
<td>1</td>
<td>3180</td>
<td>Basal</td>
<td>25</td>
<td>100</td>
<td>8</td>
<td>3</td>
<td>2.5</td>
<td>0.8</td>
<td>Breast</td>
<td>Strike</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>3400</td>
<td>Basal</td>
<td>25</td>
<td>100</td>
<td>8</td>
<td>3</td>
<td>2.5</td>
<td>1.0</td>
<td>Breast</td>
<td>Strike</td>
</tr>
<tr>
<td>Pt A</td>
<td>1</td>
<td>1030</td>
<td>UG2</td>
<td>12</td>
<td>80</td>
<td>7.5</td>
<td>2</td>
<td>1</td>
<td>1.6</td>
<td>Down dip</td>
<td>Dip</td>
</tr>
<tr>
<td>Pt B</td>
<td>1</td>
<td>1100</td>
<td>Merensky</td>
<td>30</td>
<td>100</td>
<td>10</td>
<td>3</td>
<td>1.8</td>
<td>1.4</td>
<td>Breast</td>
<td>Strike</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1306</td>
<td>Merensky</td>
<td>28</td>
<td>60</td>
<td>Stub</td>
<td>3.5</td>
<td>3.5</td>
<td>1.8</td>
<td>Breast</td>
<td>Strike</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>1200</td>
<td>Merensky</td>
<td>30</td>
<td>120</td>
<td>15</td>
<td>4.5</td>
<td>2</td>
<td>1.9</td>
<td>Breast</td>
<td>Strike</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>1240</td>
<td>Merensky</td>
<td>32</td>
<td>70</td>
<td>Stub</td>
<td>3</td>
<td>2.2</td>
<td>1.7</td>
<td>Breast</td>
<td>Strike</td>
</tr>
<tr>
<td>Pt C</td>
<td>1</td>
<td>1000</td>
<td>UG2</td>
<td>22</td>
<td>60</td>
<td>8</td>
<td>3</td>
<td>1.5</td>
<td>0.95</td>
<td>Updip</td>
<td>Dip</td>
</tr>
</tbody>
</table>
Table 2-8: Rock mechanical characteristics of the different monitoring sites.

<table>
<thead>
<tr>
<th>Mine</th>
<th>Site No.</th>
<th>Tributary area</th>
<th>In panel % extraction</th>
<th>Load (MPa)</th>
<th>PFI</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gold Mine A</td>
<td>1</td>
<td>66.0</td>
<td>98%</td>
<td>2886.8</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>46.0</td>
<td>98%</td>
<td>2794.5</td>
<td>29</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>46.0</td>
<td>98%</td>
<td>2297.7</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>49.5</td>
<td>98%</td>
<td>2739.8</td>
<td></td>
</tr>
<tr>
<td>Gold Mine B</td>
<td>1</td>
<td>48.0</td>
<td>98%</td>
<td>1231.2</td>
<td>27</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>8.4</td>
<td>88%</td>
<td>213.2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>46.6</td>
<td>98%</td>
<td>1200.3</td>
<td>16</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>76.1</td>
<td>99%</td>
<td>2115.4</td>
<td></td>
</tr>
<tr>
<td>Gold Mine C</td>
<td>1</td>
<td>12.3</td>
<td>92%</td>
<td>1051.8</td>
<td>46</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>12.3</td>
<td>92%</td>
<td>1124.6</td>
<td>52</td>
</tr>
<tr>
<td>Platinum Mine A</td>
<td>1</td>
<td>7.9</td>
<td>87%</td>
<td>245.1</td>
<td>62</td>
</tr>
<tr>
<td>Platinum Mine B</td>
<td>1</td>
<td>29.4</td>
<td>97%</td>
<td>969.8</td>
<td>43</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>11.6</td>
<td>91%</td>
<td>453.4</td>
<td>79</td>
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<td></td>
<td>3</td>
<td>13.9</td>
<td>93%</td>
<td>501.6</td>
<td>45</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>37.3</td>
<td>97%</td>
<td>1388.8</td>
<td>57</td>
</tr>
<tr>
<td>Platinum Mine C</td>
<td>1</td>
<td>9.9</td>
<td>90%</td>
<td>296.9</td>
<td>36</td>
</tr>
<tr>
<td>Platinum Mine E</td>
<td>1</td>
<td>7.7</td>
<td>87%</td>
<td>196.4</td>
<td>60</td>
</tr>
<tr>
<td>Platinum Mine F</td>
<td>1</td>
<td>6.3</td>
<td>84%</td>
<td>56.3</td>
<td>95</td>
</tr>
</tbody>
</table>
From Table 2-8, it can be seen that there is a link between the PFI and the other parameters listed. This relationship is more clearly shown graphically – Figure 2-64 to Figure 2-66 shows the relationships between the PFI and these features. Figure 2-64 and Figure 2-65 are essentially the same, showing the correlation between PFI and the nominal pillar load. In Figure 2-65 however, the data from Gold Mine C, Site 2 and Platinum Mine B, Sites 2 and 4 are excluded as these measurements were taken early in the pillar’s life-cycle when these were still stubs extending from the face.

Figure 2-64: Relationship between PFI and nominal (tributary area) pillar load.
Figure 2-65: Plot of PFI vs. nominal pillar load excluding the values for stubs.

Figure 2-66: Plot of the relationship between PFI and percentage in-panel extraction.

The above graphs show that the PFI is a good measure of the pillar behaviour in a variety of mining conditions. It shows that the pillar response does not differ between gold and platinum mines. This was also found when the geological and geotechnical conditions were analysed,
with the response being controlled by the relative strengths and positions of weaker and stronger rock, rather than absolute values.

2.5.2 Geological and geotechnical controls on pillar behaviour.

Observations of mining induced fracturing in the pillars found that it is initiated during face advance, predominantly parallel to the long axis of the pillar. This is often due to the mining configuration, where the adjacent panel lags behind. The pillar continues to fracture as mining continues away from it. The orientation and position of additional MIF sets is parallel to the free faces developed. The frequency of the MIFs also increases as the pillar becomes more and more remote from the intact reef. Higher fracture frequency in the reef indicates that the pillars are failing and not punching into the footwall. This is described below and summarized in Table 2-9, which also indicates how the PFI changes in the different stratigraphic conditions. Figure 2-67 shows these generalized pillar responses.

Where the footwall is highly fractured, it suggests that this is the weakest element in the hangingwall - pillar - footwall system. Pillars tend to bow outwards on the weakest layer of rock within them. This was found both in platinum pillars, where the movement occurred along chromitite layers and gold pillars where the movement was along shale bands. The thicker this shale band, the shallower the dip of the fractures, suggesting perhaps that more horizontal movement can be accommodated along closely spaced shale bedding planes. When a weak layer occurs on the top reef contact (TRC), the pillar is also more fractured (Table 2-9). If the immediate hangingwall above this is weak, it will fracture to the overlying stratigraphic contact and then fall out. This changes the width to height ratio of the pillar, which can cause it to fracture more. If the bottom reef contact (BRC) has low cohesion, there is more pillar fracturing. When the immediate footwall is weak, fracturing and dilation in the footwall predominates. As a result heave is increased and the footwall can slide into gullies. This is also dependant on the gullies being open to allow the footwall to slide or slab into an open space. Like the failure of the hangingwall, this also changes the width to height ratio of the pillars. This could be part of the reason why the pillars in such geotechnical areas are more likely to fail. Interestingly this decrease in the width to height ratio may also cause the pillar to fail axially, as suggested by the lower curve and diagram in Figure 2-68.

Table 2-9: The influence of stratigraphy on pillar behaviour.

<table>
<thead>
<tr>
<th>Stratigraphy</th>
<th>PFI</th>
<th>Hangingwall behaviour</th>
<th>Pillar behaviour</th>
<th>Footwall behaviour</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low cohesion TRC</td>
<td>Much higher than average</td>
<td>Less fractured</td>
<td>Increase in steeply dipping fractures and dilation</td>
<td>Less fractured</td>
</tr>
<tr>
<td>Low cohesion</td>
<td>Much higher</td>
<td>Less fractured</td>
<td>Increase in steeply</td>
<td>Less fractured</td>
</tr>
<tr>
<td></td>
<td>than average</td>
<td>dipping fractures and dilation</td>
<td></td>
<td></td>
</tr>
<tr>
<td>----------------</td>
<td>--------------</td>
<td>-------------------------------</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>BRC</strong></td>
<td>than average</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Low cohesion</td>
<td>Average</td>
<td>Less fractured</td>
<td></td>
<td></td>
</tr>
<tr>
<td>layer in pillar</td>
<td>Dilation</td>
<td>oblique dipping fractures</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Weak</td>
<td>Average</td>
<td>More fractured, fall</td>
<td></td>
<td></td>
</tr>
<tr>
<td>hangingwall</td>
<td>Width to</td>
<td>of grounds</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Weak</td>
<td>Average</td>
<td>Less fractured</td>
<td></td>
<td></td>
</tr>
<tr>
<td>footwall</td>
<td>Width to</td>
<td>height ratio decreases</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strong TRC</td>
<td>Lower than</td>
<td>Steep fractures</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>average</td>
<td>penetrate into hangingwall</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strong BRC</td>
<td>Lower than</td>
<td>Steep fractures</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>average</td>
<td>penetrate into hangingwall</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Fractures penetrating hanging and footwall
Damage to HW - especially above gully

Shearing along BRC and TRC
Slabbing off of pillar edges
Changes w/h ratio - buckling

Fracturing along pillar edge damages FW
Punching of pillar into FW, slides into gully
Continued sliding along contact, not buckling

Pre-existing discontinuity - either geological or mining induced
Limited fracturing of pillar as load is taken up by shearing
Some slabbing, but controlled by shearing on pre-existing weakness

Figure 2-67: Pillar behaviour in response to different stratigraphic and structural conditions.
If the TRC and BRC are strong, there is a large amount of damage to the hangingwall. Undulating contacts, such as is found in the vicinity of potholes on the platinum mines, prevent layer parallel movement and may be considered strong reef contacts. Selective cut mining of the reef conglomerate on the gold mines can also result in a strong TRC and BRC. These strong contacts cause pillar edge parallel fracturing of the hangingwall. Similar footwall fracturing is also likely to occur, but this is has not been observed as the footwall was often obscured by rubble.

The influence of geological discontinuities and mining induced fractures on crush pillar behaviour has also been established. It was found that if there are suitably oriented discontinuities, either pre-existing geological features such as joints or mining induced shear fractures that formed before the pillar was cut, much of the stress that should have resulted in the fracturing of the reef can actually be accommodated along these discontinuities. The body of the pillar would thus be prevented from crushing and yielding, thus resulting in a lower PFI. As the load increased with further mining, these discontinuities become unable to move and yield, resulting in a stress build-up, which could not be relieved through movement of the fractured reef zone. In several cases this resulted in the strain-bursting of crush pillars. This incorrect crush pillar behaviour may be likened to the shear failure of a UCS specimen (Figure 2-68).

In the different stratigraphic and structural domains described above, there does not appear to a possibility of the pillar crushing to form a blocky rock-mass analogous to the UCS specimen indicated in the upper curve in Figure 2-68.

![Figure 2-68: Change in nature of failure of different UCS specimens. The upper curve and diagram represents crush / yield failure, the middle curve is theoretical behaviour of crush pillar and lower diagram is similar to observed crush pillar behaviour. Modified after Ryder and Jager (2002), Figure 2.2.4.](image-url)
Closures of between 30 and 50 mm per week were measured at different sites, from deep level gold mines to shallow platinum mines. Where crush pillars fractured as designed, the closure appeared to be relatively constant across the panels. This indicates that when these pillars are acting correctly, the hangingwall rock mass acts as a single beam, usually parting on stratigraphic weaknesses. Borehole logging and mapping of pillar holings created an extensive database of fracture characteristics of crush pillars across many different reefs, depths and ages. The fracturing of other pillar types was also quantified to allow the comparison of crush pillar behaviour with these different pillars. From this database the Pillar Fracture Index (PFI) was developed. The PFI provides a single number that encapsulates all the characteristics of the pillar and thus allows the easy comparison of different pillars. It was found that all crush pillars had Pillar Fracture Indices of less than 45. Slight variations in the PFI can be ascribed to variations in the geotechnical conditions and age of specific types of pillars. For example low PFI values can be due to both abundant geological discontinuities (such as around potholes) or the fact that the area was mined out long ago. PFI values are similar in different mines on different reefs, where the age of the pillars are similar, despite the fact that these pillars were formed on different reef horizons. Pillars that initially have a PFI of a yield pillar, dropped to a crush pillar PFI after several weeks. It appears as though there is a continuum in rock mass behaviour (including fracturing, closure and pillar dilation) between crush and yield pillars, highlighted by the change in PFI values with time. Even poorly cut grid pillars can act as either crush or yield pillars.

2.6 Conclusions from underground studies

From the underground studies, the following conclusions can be drawn on crush pillar behaviour:

1) Crush pillars are not affected by seismicity from distant events. No correlation between the microseismicity (i.e. fracturing) of the pillar and larger distant events was found.

2) Crush pillars are not completely formed ahead of the mining face. Both seismic and PFI evidence show that fracturing continues for tens of metres behind the face.

3) The fracturing of a crush pillar is dependant on the mining history, with fractures developing parallel to the free edge and wrapping around the pillar as each subsequent side is developed.

4) Where low cohesion reef contacts are present, the fracturing of crush pillars is like that of a UCS specimen compressed with frictionless platens (see Figure 2-68).

5) Where weak hangingwall is present, fallout can change the width to height ratio of the crush pillar and thereby cause an increase in fracturing.

6) Where a weak footwall is present, this can slide into gullies allowing the dilation of fractures in the pillar.

The PFI has been found to be an accurate measurement of pillar conditions since there is a strong relationship between the PFI and pillar load and percentage in-panel extraction. There may be a different fracture rate (determined from PFI) for crush pillars in different areas, and there is also potentially a correlation between the rate of PFI change and pillar closure.
3 Numerical modelling

3.1 Introduction

Numerical modelling is necessary to fully understand crush pillar behaviour under different loading conditions. This section of the report presents the three-dimensional, linear elastic modelling carried out to assess the loading of in-panel pillars with respect to the mining, to understand the effect of the surrounding environment, and to estimate the load bearing capacity of a crush pillar. It also defines in detail the results of the distinct element / finite difference code (UDEC) investigations into pillar behaviour.

The underground studies of crush pillars as part of this project appeared to indicate two interesting and previously unconsidered aspects of behaviour of in-situ crush pillars. Firstly, repeated borehole mapping revealed that open fractures in the edge of the pillar tend to disappear with time. Fractures mapped on one visit are not observed on a subsequent visit. The second unexpected observation made during the course of underground monitoring is the frequent existence of a zone of high intensity fracturing in the centre of the pillar. If this fracturing does indeed exist then it indicates that the very core of the pillar is failing: this seems totally counterintuitive.

Using a distinct element / finite difference code (UDEC), the incongruous pillar behaviour described above was investigated. A major objective was also to attempt to quantify the relationship between pillar deformation and PFI; and from this, to link PFI to the stress-strain behaviour of a crush pillar. These factors were analysed against the underground monitoring data.

A different approach to the normal continuum ‘strain-softening’ modelling of crush pillars is attempted here. In this approach, no complex constitutive model is used with parameters that need to be manipulated. Rather, a mesh is created composed of very small triangular blocks separated by joints that are controlled by a simple Mohr-Coulomb failure model. As with the continuum models, it is accepted that the results are mesh dependent but in this case the mesh dependency has some direct physical meaning – namely the potential in-pillar fracture positions and corresponding PFI values.

3.2 MINSIM modelling

The objective of preliminary linear-elastic numerical modelling (using MINSIM) was to gain an understanding of the possible crush pillar stress loading from initial cutting of the pillar until it is in the back area. The pre-processor software RExlgrid and RExl5ln was used to produce the input files for the MINSIM type jobs.

Early versions of MINSIM were developed at the Chamber of Mines Research Organization in the early 80’s. Successive versions of MINSIM-D have been developed by the CSIR Natural
3.2.1 Applications, assumptions and limitations of MINSIM

The MINSIM solution engine is built around a linear elastic boundary element solution based on the displacement discontinuity method. It provides an optimised three-dimensional solution engine for solving large scale; underground, tabular ore body mining problems in the fastest possible time. Traditionally, its market has been the gold mines of the South African Witwatersrand Basin, however, it has been found increased application within the South African platinum and coal mining industries.

Because the MINSIM solution relies on the assumption of linear, elastic, homogeneous and isotropic rock mass behaviour, it is constrained by the limitations of any linear elastic analysis program in that it cannot model rock failure and plastic deformation.

3.2.1.1 MINSIM solution for tabular excavations

In the MINSIM method of analysis, all stresses in the medium are determined by the relative movement between the roof and the floor of the excavations – the so-called displacement discontinuity. The displacement discontinuity is a vector quantity whose components are conveniently resolved along two axes in the place of the excavation and a third axis normal to the plane of the excavation. The components in the plane are termed ‘rides’ and the normal component is termed the ‘convergence. In general, all components will vary over the excavation and are solved to satisfy specified stress boundary conditions in the excavation. The mining outlines on a single reef plane are described by an array of 64 X 64 grid elements, each of which are designated as ‘mined’ or ‘solid’.

Tabular excavation mining problems are represented by the analysis of thin, sheet-like displacement discontinuity sheets that divide the mined area into a regular array of elements. The actual problem is solved by superimposing the grid of the square elements over the mined areas. Planar groups of 64 X 64 elements (termed “windows”) are used to provide a detailed resolution for most excavation outlines. Each element in the window is assigned a status of ‘mined’ or ‘solid’. A mining problem is generally represented by several interacting windows of 64 X 64 elements. Each mining problem is considered to comprise a set of \( n \) windows which are oriented to cover the mining areas of interest. The solution strategy is to solve each of the \( n \) windows in turn. Prior to the solution of the current window, the external stress influences due to the most recently estimated movements on all other windows are computed at each element of the window, using the stored kernel table and lumping techniques described by Napier and Stephansen (1987). The current window solution is then iterated to a specific tolerance. The maximum residual error occurring during iteration of each window is saved and compared to a specified tolerance after the solutions on all \( n \) windows have been updated. The entire cycle is then repeated until convergence is achieved.

3.2.2 Investigation site

The global mine plan of Platinum Mine B, used for the MINSIM modelling is shown in...
Figure 3-1. Figure 3-2 is a zoomed-in view showing the position of Site 3 and the pillar that was monitored over a 10 weeks period. Also shown are the face positions used in the model. The extent of the global mining area modelled is about 1000 metres on strike and 800 metres on dip. The global mining area was digitised to be used in the three-dimensional linear elastic modelling. A dip of 12 degrees and a depth of approximately 1175 meters below surface were used in the model.

*Figure 3-1: Mine plan of the overall mining area. The dashed line represents the zoomed area in Figure 3-2 below.*
3.2.3 MINSIM modelling procedures

3.2.3.1 Calculation of APS in MINSIM

The computation of average pillar stress is complicated by the use of an edge correction (Lightfoot and Maccelari, 2000). If no edge correction is used, average pillar stress can be accurately estimated, even if the pillar is one element wide, provided the pillar and element coincide. The edge correction used in MINSIM enables the element stress to be accurately represented at its central point. However, if the element lies at the edge of the mined-out region, the stress associated with the central element will be an underestimation of the actual stress. For this reason, average pillar stress is generally underestimated. The extent of underestimation is related to the size and number of elements used in the computation. It is necessary to have at least four elements across the width of a pillar in order to achieve reasonable approximations of stress for the face elements.

The formulation of partially mined elements is described by Napier and Stephansen (1987). In practice, MINSIM allows elements to be considered partially mined in increments of 5%. MINSIM does not publish stress values for partially mined elements; the output from the program reports that the stress on all partially mined elements is zero. For this reason, the partially mined elements around the edge of the pillar make no contribution to the total stress acting on the pillar. Hence, when the average pillar stress is calculated by dividing total stress...
by current pillar area, the result will be too low. The smaller the percentage mined of any given element, the larger its contribution to the overall error. When an element is 95% mined, its actual area is very small and thus the contributing error is small. However, when an element is say 5% mined, it will have a significant area but will be reported as carrying no load and thus the error contribution will be large.

MINSIM modelling was undertaken with the use of RExlgrid and RExl5 (Rock Engineering in Excel) programs developed by Itasca Africa (Pty) Ltd. RExlgrid is a pre-processor that relies on creating specialised Excel worksheets that “understand” the concept of an on-reef mining sheet. This sheet is automatically formatted as a sheet of square cells that represent individual displacement-discontinuity (DD) elements. Arbitrary mining shapes can be defined and backfill can be assigned to both to individual cells or groups of cells. Mining steps are catered for. The mining patterns drawn on a single Excel worksheet are naturally tiled. This pre-processing system has been designed to work primarily with MINSIM-D and MINSIM-W type 64 by 64 element DD sheets but can easily be extended up to 256 elements on strike by 65536 elements on dip. RExlgrid was used to produce the input files for all the MINSIM type jobs conducted in the analysis described in this report.

RExl5 is a generic modelling post-processing facility that contains a specialised MINSIM post-processing facility. The implementation of the RExl5 post processing facility for MINSIM involves resizing Excel worksheet cells to be small squares and then using each of these cells to represent either a MINSIM on-reef element or an off-reef field point. Once the MINSIM data is imported into Excel, various rock engineering analyses can be performed using RExl5.

### 3.2.3.2 Rock mass and field stress parameters

Rock mass parameters and the magnitude of the components of the virgin stress tensor are based from underground fracture orientations and measurements by Groundwork personnel. A Young’s Modulus of 70 GPa and a Poisson’s Ratio of 0.20 were used in the model; and the stress gradients (TXX, TYY and TZZ) were all set at 0.03 MPa / m (the modelled pillar stresses are actually extremely insensitive to all of these particular variables). The reef parameters are given in Table 3-1.

**Table 3-1: Reef Parameters**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Global model element size</td>
<td>2 m</td>
</tr>
<tr>
<td>Local model element size</td>
<td>0.5 m</td>
</tr>
<tr>
<td>Mining Steps</td>
<td>3</td>
</tr>
<tr>
<td>Reef Dip</td>
<td>12°</td>
</tr>
<tr>
<td>Stoping Width</td>
<td>1 m</td>
</tr>
</tbody>
</table>
3.2.3.3 RExlgrid models

Figure 3-3 shows a discretised model from RExlgrid. The model covers a mining area of 4 by 6, 64 by 64 element displacement discontinuity (DD) sheets using 2 metre square elements. This covers a total modelled area of 512 x 768 m. Although this model gives a good indication of anticipated stresses at various stages of mining on the reef horizon, a 2 metre grid size is not necessarily a good indication of the loading condition of the in-stope crush pillars.

It is also impractical to model the full scale mining scenario with a smaller grid size such as 0.5 metres, as this would take too long to run. It is more sensible to reduce the areal extent of the numerical model. The smaller area of mining in such a model means that the actual pillar loads on-reef are significantly reduced as are the stress, strain and displacements off-reef and thus needs to be accounted for in a suitable manner. Two types of calibration models, a global and local model (Figure 3-4 and Figure 3-5 respectively), were set up to assess the loading of the in-stope crush pillars as mining progresses past the pillars. The maximum panel convergences of both models were compared and an adjustment was made on the applied field stresses on the local model to account for the large extent of mining. This is possible as the model is essentially linearly elastic, and so is not adversely affected by this adjustment in any significant manner.

![Figure 3-3: Initial Global model with in-stope crush pillars.](image)

The global calibration model covers the same area as the base model in Figure 3-4 with in-stope crush pillars removed. The local model covers a mining area of 4 x 5, 64 x 64 element DD sheets using 0.5 metre square elements. This covers a total area of 128 x 160 m.

The maximum panel convergence indicated in the global model without in-panel pillars is around 97 mm. The unadjusted local model of this scenario produced the relevant in-panel convergence values of around 76 mm. As the in-panel convergence in the global calibration...
model is 1.3 times that predicted by the local model, it is clear that the field stress on the local model must be adjusted by the same factor to produce in panel convergence values in the local model consistent with those of the global model.

Figure 3-4: Global calibration model without in-stope crush pillars.

Figure 3-5: Local calibration model without in stope crush pillars.
Figure 3-6: Final Local model showing in-stope grid pillars.
Figure 3-7: The on-reef convergence field for the global calibration model. The dashed line shows the location of the section line used for graphing convergence profile below.

Figure 3-8: Convergence profile along the panel in the global calibration model
Figure 3-9: The on-reef convergence field for the local calibration model. The dashed line show the location of the section line used for graphing the convergence profile below.

Figure 3-10: Convergence profile along the panel in the local calibration model
Figure 3-11: The on-reef convergence field for the calibrated local model. The dashed line shows the location of the section line used for graphing the convergence profile below.

Figure 3-12: Convergence profile along the panel in the calibrated local model
3.2.3.4 Average pillar stress results

The purpose of this MINSIM analysis has been to identify the loading conditions in the in panel crush pillars due to the extent of mining around the area. Figure 3-13 below shows a plot of on-reef stress field in the same panel over which underground monitoring was conducted with the pillar where detailed monitoring occurred indicated by the dashed line. The Average Pillar Stresses (APS) of the highlighted pillar is summarized in Table 3-2. Figure 3-14 is a plot of on-reef average pillar stresses at the last mining step.

![Figure 3-13: The on-reef normal stress field for the local model](image-url)
Figure 3-14 On-reef average pillar stress in MPa for the local model at mining step 3

Table 3-2 Summary of modelled APS of the monitored pillar.

<table>
<thead>
<tr>
<th>Mining step</th>
<th>APS (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>185</td>
</tr>
<tr>
<td>2</td>
<td>288</td>
</tr>
<tr>
<td>3</td>
<td>314</td>
</tr>
</tbody>
</table>

Empirically, from an elastic modelling point of view, these APS values make sense but are much higher than the expected residual crush pillar stresses (10 to 25 MPa) observed by Roberts et al. As an elastic solution, MINSIM does not take into account the strain-softening behaviour of
the pillars due to failure, hence the pillar stresses increase linearly as the demand on them increases.

A second run was made by removing all the back area pillars and replacing these with “backfill soup” that smeared out the expected residual stresses across the back-area. The smearing of the back-area with “backfill soup” is seen as a way to mimic the crushed pillars that would have been at a low load bearing capacity in this area. It does not mean that actual backfill was being modelled. The local pillars in the panel were kept intact. The “backfill” stress was kept at a constant 0.75MPa, 1MPa and 2MPa in the backfill stress-strain behaviour. This made very little difference in the average pillar stress values. The linear-elastic modelling therefore does not represent the residual stress of the crush pillar system. Modelling of the in-panel crush pillars as an elastic material thus provided only a very marginal additional insight into the expected levels of “peak” stress in the pillars. From the modelling results, peak APS varies from 90 MPa (as a pillar stub at the face) to over 300MPa in the back-area where the pillar will in reality have crushed.

### 3.3 UDEC modelling

#### 3.3.1 Introduction

The use of UDEC as a means towards explicitly modelling pillar fracturing and PFI behaviour was motivated in Section 3.1. The FISH language in UDEC has been used extensively during the course of the research presented in this report. After creating the model of a crush pillar as a highly discontinuous assemblage of blocks and subsequently subjecting it to some kind of loading it is possible to perform a Pillar Fracture Index analysis on the model analogous to that performed physically.

#### 3.3.2 The Discontinuum Model

The numerical modelling simulations described here were all undertaken using the Itasca Consultancy Groups 2D modelling program UDEC (Universal Distinct Element Code). For a full description of UDEC see Numerical Modelling of Mine Workings, Second Edition, Volume II (SIMRAC, 2000). All analyses performed here were undertaken with the command-line Version 4.0 of UDEC. Instructions were prepared for simulations in the form of ASCII data files.

#### 3.3.3 The UDEC Program

A UDEC block assemblage is created by defining all existing and potential rock fractures and joints before the simulation begins. The fractures and joints can be given properties that can make them either open or closed (intact) at the start of a simulation. Intact fractures can “break” during simulations as a consequence of applied loads resulting in the creation, or growth, of a fracture. The rock between the explicitly predefined joints and fractures is considered to be a locally continuous rock mass. This rock mass can be an unbreakable elastic material or it can be an elasto-plastic material that is allowed to fail as a local continuum.
The joints and fractures can be assigned material properties based on a number of well known constitutive laws but the only constitutive law for joints and fractures used in the work reported here is a simple elasto-plastic model with a Coulomb strength law. In all the models discussed here the Coulomb strength is controlled by only three parameters, namely slip friction angle, cohesive strength and tensile strength: no dilation angle was used in any of the simulations. Furthermore, once a fracture has passed its elastic yield point and failed in some way, a flag is set to ensure it will remain failed for the rest of the simulation.

The rock mass between the joints forms blocks that can be rigid, elastically deformable or plastically deformable. In the simulations described in this report the blocks are considered to be elastically deformable: they cannot yield plastically. In UDEC it is usual to sub-divide each elastic block into a number of constant strain finite difference triangles which will allow the block to deform when subjected to loading. In the analyses covered here the rock blocks themselves are very small triangles so it is not necessary to sub-divide them into multiple finite difference triangles: each block is composed of a single finite difference triangle. This allows the blocks to deform in a relatively simple manner but more importantly it allows the blocks to support a finite value of stress. If the blocks were left as rigid triangles then it is not possible to calculate the value of stress acting on the block as, by definition, a material with no strain has no stress.

3.3.4 The Block Assemblage

For the purpose of the analyses presented in this report a single crush pillar is represented as a conglomeration of many thousands of triangular UDEC blocks. This approach to the modelling of hard rock mining was first presented by Lightfoot (1993) in the context of modelling the rock mass ahead of a deep level gold mine. This work was extended by Lightfoot, et al (1994) to simulate the potential for face-bursting of deep level gold mine stopes. In the current scenario, the pillar is loaded from above and below through intact rock blocks (Figure 3-15). The inelastic properties of the rock mass of the pillar are prescribed in terms of a limited number of simple Mohr-Coulomb parameters. The strain softening/hardening behaviour of the pillar is produced as a consequence of the highly discontinuous (blocky) nature of the rock mass rather than by some complex, multi-parameter, constitutive law.
Field evidence indicates that the average fracture spacing is 60mm and that the preferential fracture orientation is at a dip of 70 degrees. Based on this a model was developed with a smaller fracture spacing and joints oriented at plus and minus 70 degrees, as well as vertical and horizontal (Figure 3-16). The spacing between the vertical joints in the model is a distance chosen to be less than the average fracture spacing measured underground. The dipping and horizontal joint spacings are chosen to form discontinuities that produce coincidental intersections required to produce triangular blocks throughout the model. This is done to produce a pseudo-homogeneous material throughout the pillar.
The behaviour of the joint discontinuities between blocks is controlled by a simple Mohr-Coulomb constitutive law. There are only three parameters that control this constitutive model, these being the surface friction angle, the cohesive strength and the tensile strength of the joint. If either the cohesive strength or the tensile strength of the joints are given finite values then these joints do not represent actual fractures: they represent potential fracture planes.

The triangular blocks themselves are stiff but deformable elastic blocks each with six independent degrees of freedom that cannot fracture into smaller blocks. The blocky discontinuum covers the 1.6 metre high pillar horizon and extends for 0.8 metres into both the hangingwall and the footwall. The model itself is controlled by only four independent variables, the first being the potential vertical joint spacing, the other three being the joint friction angle, cohesive strength and tensile strength. These parameters control the micro-mechanical behaviour of the rock mass, and there is no direct way to predict the resulting macro behaviour of the pillar. It is not possible (nor would it be desirable) to work out, in advance, values of input parameters to directly control the behaviour of the pillar. The only way to determine the influence of the input parameters on the overall behaviour of the pillar is by experimentation. Such experiments provide an environment in which it is possible to learn a great deal about how a pillar behaves based on simple rock strength parameters and well defined loading conditions. The down-side to this experimental approach to modelling of pillar behaviour is that the computer simulations require a considerable amount of time to run.
3.3.4.1 Joint Spacing and Block Size

A number of different values of vertical joint spacing have been implemented, varying from 100mm to 12.5mm, producing block assemblies that vary between roughly 4 000 blocks up to almost 52 000 blocks. The run times to produce useful results for these block assemblies ranges from about two hours to more than two days depending on the number of blocks.

3.3.4.2 Boundary Conditions

The idea was to simulate a rib crush pillar at a depth of approximately 1 000 metres. In order to achieve this, typical gold mine compressive field stresses of 27 MPa were applied in the vertical direction and half of this in the horizontal direction (a k-ratio of 0.5) before mining adjacent to the pillar. The vertical sides of the model were fixed in the x-direction, effectively making these edges axes of symmetry. The model was then cycled to consolidation.

Figure 3-17: Details of the initial block assembly and boundary conditions. Arrows on the boundaries indicate applied stress or displacement boundaries and circles indicate roller boundaries, that is, axes of symmetry.
After consolidation, the ground to either side of the actual pillar was removed to leave a pillar of height 1.6 metres and a width of 3.0 metres. In the case of the early simulations, the model was subsequently cycled for a reasonable length of time under the stress boundary conditions. In the later models the upper and lower stress boundaries were replaced by constant velocity boundaries and the model was subsequently cycled for a fixed amount of “problem” time.

3.3.4.3 Constitutive Models

A very simple constitutive model was utilized - the rock blocks were represented as completely elastic material that could deform but not fail. The joints were represented by a simple Mohr-Coulomb model covering an areal length of the joint. The Mohr-Coulomb model was described in terms of the slip friction angle, the cohesive strength and the tensile strength of the contacts representing the joint. The constitutive model was chosen such that when a joint yields plastically, a fracture flag is set to ensure that the cohesive and tensile strengths are maintained at zero for the rest of the duration of the simulation. All strain softening or hardening behaviour exhibited in the models is a consequence of the highly discontinuous nature of the block assembly.

3.3.4.4 Material Properties

The rock mass is modelled as a homogeneous material with consistent (‘gold mine’) properties throughout. The rock blocks were modelled as stiff elastic blocks with a density of 2700 kg/m$^3$ and a bulk and shear modulus of 32 GPa and 25 GPa respectively (i.e. $E = 59.5$ GPa and $\nu = 0.19$). The joints between the blocks were modelled with a simple Mohr-Coulomb constitutive model with a friction angle of 30 degrees and a cohesive strength that varied between 5 MPa and 20 MPa for particular simulations. In all cases to date the tensile strength of the joints has been set to zero to reduce the number of material parameters that can effect the current simulations. At some time in the future it is hoped that there will be an opportunity to test the effects of a finite tensile strength of the joints. There is no dilation on any of the joints at this time for the same reason.

3.3.5 Model Development

The earliest experiments relating to this project involved three different analyses. The first set of analyses involved varying the block sizes in the models, the second set of analyses involved fixed block sizes with different values of cohesion using stress boundary conditions and the final set of analyses used displacement boundary conditions.

3.3.5.1 Block Size

Initially the goal was to assess the effect of the block size on the potential pillar behaviour. A number of snap-shots of pillar deformations are presented for comparative purposes (Figure 3-18). In the models with 13 000 blocks or more, the pillar does start fracturing from its edges but as the fractures develop further into the pillar those towards the edge close up and the blocks tend to clump: this appears to be consistent with the field observations.
Figure 3-18: Early experimental block assemblies with different numbers and sizes of blocks subject to force loading.
3.3.5.2 Stress Loading with Different Values of Cohesion

Initially it was decided to use the large 52 000 block model to test the pillar behaviour subject to stress boundary conditions for various cohesive strength values of the pillar “joints”. Joint cohesive strengths of 5 MPa, 6 MPa and 10 MPa were used in the experiments. The applied virgin (field) stresses were increased step-wise from their original values of 27 MPa in the vertical direction and 13.5 MPa in the horizontal direction. These values of stress load are obviously an underestimation of the actual field loads that such a pillar would be subjected to in practice. The simulations were allowed to run until either the pillar failed plastically and exhibited a reasonable degree of yield or it was found to be stable. It should be noted that this set of simulations was extremely time consuming with some runs taking up to two and a half days.

In the case of the 5 MPa cohesive strength, the pillar failed at an internal vertical load (APS) of about 65 MPa and continued to yield at this load (Figure 3-19). The simulation was run to a point where approximately 60mm of convergence has occurred on the pillar. A significant amount of fracturing is observed on both edges of the pillar as would be expected. If gravity were imposed on the model, then this might well result in spalling at the edges of the pillar. What is of particular interest in this run is the shear band that develops through the centre of the pillar - possibly the core fracturing seen in the field mapping of the real pillar. With an increase in cohesive strength of the pillar joints to 6 MPa, the pillar was seen to yield at around 70 MPa (Figure 3-20). This simulation was run for twice the length of the previous simulation to a total pillar convergence of around 120mm. Initially, after failure, the pillar shed load but after further yield the load began to climb to around 75 MPa. With the increase in convergence more damage is seen from the edges of the pillar towards the centre, but the central shear band is still clearly visible.

Figure 3-19: Assembly with approximately 52 000 blocks subject to a force loading with a cohesive strength of 5 MPa – the pillar is in a state of plastic yield. The graph shows stress (Pa) against hangingwall displacement (m).
Figure 3-20: Assembly with approximately 52 000 blocks subject to a force loading with a cohesive strength of 6MPa – the pillar is in a state of plastic yield. The graph shows stress (Pa) against hangingwall displacement (m).

With the cohesive strength of the pillar joints increased to 10 MPa, the pillar was seen to become stable under the applied loading (Figure 3-21). The only fracturing that occurs in the pillar is at the very edge and the pillar as a whole does not exceed its elastic strength.

Figure 3-21: Fractures in a block assembly with approximately 52 000 blocks subject to a force loading with a cohesive strength of 10MPa and the associated deformation-load curve – the pillar is in a stable state.
The stress loading experiments have produced some interesting results to date and probably need to be considered in more detail. It is apparent the applied stresses are definitely too low and that higher values of joint cohesion should be used. Unfortunately time constraints prevent more immediate work in this area but it would probably be a good idea to revisit this area of analysis at some time in the future. At this point, it was decided to leave the stress loading models with 52 000 blocks and concentrate on models with a reduced number of blocks subjected to displacement loading.

3.3.5.3 Displacement Loading

At this point, a mistake was discovered in the UDEC command used to create the vertical joints in the model. This caused every second vertical joint to be missed, with the consequence that a large number of blocks in the mesh were not being split in two as intended. This problem was rectified resulting in a block assembly of roughly 17 000 blocks.

In the case of the displacement loading models the boundary conditions were as described above but the joint cohesive strength was set at a more realistic 20MPa. A displacement rate was set such that a simulation could produce roughly 20mm of convergence on the pillar in about 12 hours of run time. It may be possible to increase this displacement rate to reduce run times and produce comparable results but this needs to be tested.

The displacement loading of the pillar produces significantly different results from those indicated by the stress loading of the pillar (Figure 3-22). Most of the fracturing occurs from the edge of the pillar inwards. The simulation was run with “movie” mode active and the resulting movie shows some interesting behaviour that it is really not possible to convey in the format of this report. Shear bands are seen to develop towards the centre of the pillar but not nearly so dominantly as in the stress boundary models.

The overall behaviour of this pillar is towards that of a "crush" pillar rather than a "yield" pillar observed in the previous experiments. The pillar loads elastically and then fails and sheds load. After about 8mm to 9mm of convergence it reloads slightly again up to around 17mm of convergence when it starts on a load shedding trend. The trends in the deformation-load characteristics are similar to those usually attributed to typical crush pillar behaviour although the details are considerably more complex.
3.3.5.4 Loading Conditions

A stope convergence in the holing between two crush pillars of around 24mm was measured at the main field site. Initially this value was used to load the pillars in most of the simulations. However, this value might be rather conservative as it does not account for the inelastic convergence that accompanies the blast or any additional convergence that occurs after this but before the convergence-ride stations were installed underground. Review of the MINSIM modelling presented elsewhere in this report indicates that initial elastic stope convergence in the vicinity of the pillar at the field site may be of the order of 15mm to 20mm. Hence, the total convergence for the later simulations was pushed to 40mm.

3.3.5.5 Horizontal Compression in the Hangingwall & Footwall

Due to constraints on the size of the model, the stoping span being modelled is only three metres. This is clearly an inadequate span and the horizontal stresses in the hangingwall beam remain very high as they are unable to relax.

The high horizontal compression in the hangingwall is a direct consequence of the loading history of the model. In order to simulate a pillar at roughly 1 000 metres depth the intact model was loaded with a vertical compression of 27MPa (Figure 3-23) and a horizontal compression that was half of this (Figure 3-24). Mining on either side of the pillar was then simulated by deleting areas of blocks. Finally, the displacement boundaries were applied to the top and bottom and the model was allowed to cycle. These initial stresses applied to the model result in the locked in stress in the hangingwall (Figure 3-25 and Figure 3-26). If the mined spans were larger and mined in a more stepwise fashion then the horizontal stresses would probably

Figure 3-22: Fracturing in a 20 MPa cohesive strength pillar after roughly 50 mm of convergence with the associated deformation-stress curve.
dissipate. Unfortunately, such a model would take many days to run and is impractical at this stage. Alternatively, it is possible to start with a lower applied stress field.

It was decided to test this hypothesis by reducing the horizontal component of stress while maintaining the vertical stress. Rather arbitrarily it was decided to apply a horizontal stress of 1MPa; the actual quantitative results of this experiment are not important, it is simply a test of the prediction of the hypothesis. As a result of using the initial 1MPa horizontal stress (Figure 3-27) the resultant locked in horizontal stress is much reduced (Figure 3-28). More importantly, the rock in both the hangingwall and footwall is seen to form shear bands (Figure 3-29): the action is no longer isolated to the reef horizon. This new behaviour seems to be more consistent with what is observed in reality and would indicate that there are problems with the current boundary conditions controlling the model.

Figure 3-23: Vertical stress through the pillar and immediate hangingwall and footwall before any displacement loading.
**Figure 3-24**: Horizontal stress through the pillar and immediate hangingwall and footwall before any displacement loading.

**Figure 3-25**: Vertical stress through the pillar and immediate hangingwall and footwall after displacement loading.
Figure 3-26: Horizontal stress through the pillar and immediate hangingwall and footwall after displacement loading.

Figure 3-27: Applied horizontal stress of 1MPa through the pillar and immediate hangingwall and footwall before any displacement loading.
Figure 3-28: Reduced horizontal stress after an initial applied 1MPa horizontal stress through the pillar and immediate hangingwall and footwall after displacement loading.

Figure 3-29: Fracturing is seen to propagate into the hangingwall and footwall when the initial applied horizontal stress is reduced to 1 MPa.
3.3.5.6 Defining Fractures

All of the plots shown above are simply boundary plots that show both the external and all internal boundaries within UDEC. It was found that a better visual representation of fracturing was produced by combining boundary plots with plots of open fractures. The shear and shear plots were not used during the production of this report.

3.3.6 Repeatability

During the course of the analyses performed as a part of the work presented in this report an intriguing, and perhaps disturbing, phenomenon was observed with the blocky rock mass simulations. It was found that by changing only the cycle increments for a simulation the final results were altered. Consider the situation where a model is cycled in five increments all of 0.05 seconds of simulated time, this might be expected to produce the same results as a simulation that is cycled in twenty five increments of 0.01 seconds of simulated time. This is not the case (Figure 3-30). In order to avoid problems of repeatability as a consequence of this phenomenon it is important to use the same cycle increments for all simulations.

![Figure 3-30: Two sets of fractures produced from identical models that are given different CYCLE increments. The block shown in B was given five times more CYCLE increments than that shown in A: the final number of cycles and problem time is identical for both cases.](image)

In order to ensure compatibility when analyzing the results of subsequent UDEC simulations the decision was made to fix the block size, geometry and number of blocks in a simulation from this point on. The block assemblages with a final total (after mining) of 17 276 blocks, including the platen blocks, were selected as a compromise between a sufficiently large number of blocks and acceptable computer run times.
3.3.7 Pillar Strength

The strength parameters applied to the UDEC contacts are not directly related to the rock mass strength parameters measured in the laboratory. For the purpose of the type of analyses discussed here, the joint (or contact) strength parameters can be seen more as micromechanical bond strengths within the material. However, it is important to be able to relate the bond strength parameters to rock mass properties measured in the laboratory. The slip friction angle and the cohesive strength of the contacts are related in some way to the uniaxial compressive strength of the rock mass but the bond tensile strength is an altogether more difficult issue. The pillar width also plays an important role in how the pillar will behave when subject to load.

3.3.7.1 Pillar Width

The width of the pillar obviously has an effect on how it will perform, with conventional wisdom suggesting that wider pillars will support more load. Unfortunately, in the course of this work it was only possible to test this assumption with a single simulation due to time constraints. A 4 metre wide pillar was modeled with a ubiquitous tensile strength of zero and a rock mass UCS of 125MPa and compared to a 3 metre wide pillar with the same rock mass properties. The model definitely indicates a much more extensive core in the 4 metre wide pillar than in the 3 metre wide pillar (Figure 3-31).

![Figure 3-31: The effect of increasing the pillar size on the fracture pattern within the pillar. A) The default 3 m wide pillar, B) a 4 m wide pillar: the tensile cutoff is set at zero in both cases.](image-url)
### 3.3.7.2 Joint Shear Strength

It is possible to assess the potential strength of the individual joints in terms of the values of their specific Mohr-Coulomb parameters from the equation for the shear strength $\tau$ of a joint given by

$$\tau = c_0 + \sigma_n \tan(\phi)$$

where $c_0$ is the cohesive strength of the joint and $\phi$ is the angle of slip friction of the joint. The value $\sigma_n$ is the component of stress acting normal to the joint as a result of the applied field stress. If the absolute value of the shear component of stress $\sigma_s$ acting along the direction of the joint is greater than the shear strength then the joint will slip and fail plastically. The normal and shear components acting on a surface can be computed from the equations given by Crouch and Starfield (1983) for a fracture whose normal is oriented at an angle $\alpha$ to the $x$-direction.

$$\sigma_s = (\sigma_{xx} - \sigma_{yy}) \sin \alpha \cos \alpha - \sigma_{xy} (\cos^2 \alpha - \sin^2 \alpha)$$

$$\sigma_n = \sigma_{xx} \cos^2 \alpha + 2\sigma_{xy} \sin \alpha \cos \alpha + \sigma_{yy} \sin^2 \alpha$$

Assuming a vertical stress of 27 MPa, the shear stress and normal stress acting on a fracture at the edge of the pillar dipping at seventy degrees to the horizontal are minus 8.68 MPa and plus 3.16 MPa respectively. A cohesive strength of 20 MPa alone for the fracture would be able to overcome the shear stress acting on the fracture and the fracture would not fail. Indeed, any such fracture with a cohesive strength greater than 5.52 MPa would not fail in shear. The shear and normal components acting on a fracture inclined at seventy degree to the horizontal are minus 4.34 MPa and plus 15.079 MPa respectively. A fracture with no cohesion and an angle of slip friction of thirty degrees would have a shear strength of 8.7 MPa under such loading conditions.

### 3.3.7.3 Joint Tensile Strength

It is difficult to know what value to assign the tensile strength of the joints. In the case of a simple Mohr-Coulomb failure criterion the tensile strength $\sigma_T$ is derived from the simple linear Mohr-Coulomb intercept given by Brady & Brown (2004) as

$$\sigma_T = \frac{2c \cos \phi}{1 + \sin \phi}$$

In practice, this always gives values that are higher than those measured in the laboratory. For this reason, a tensile cutoff is often applied at a value less than that given by this simple Mohr-Coulomb tensile strength intercept. Griffith crack theory indicates that this value should be one eighth of the uniaxial compressive strength of the rock, whereas others suggest it should be one tenth of the UCS.
The application of a tensile strength cutoff that is one tenth the UCS of the rock mass to all contacts in the UDEC simulation results in unrealistic behaviour of the modeled pillar. As a rock mass that contains any discontinuities is essentially incapable of sustaining applied tensile loads it is generally considered prudent to set the tensile cutoff to a value of zero (Brady and Brown, 2004). This may be acceptable when the material is modeled as a continuum. In the case described here it would be difficult to argue that all contacts, or bonds, should have zero tensile strength.

A series of simulations were performed in which the value of the tensile strength of all bonds in the block assembly was varied from zero to 12.5 MPa (one tenth the UCS of the rock fabric) and the resulting fracture patterns were observed (Figure 3-32). It is clear from the tensile strength analysis that a ubiquitous tensile strength for all contacts of 12.5 MPa produces extremely limited fracturing within the pillar. In fact, in order to produce any reasonable amount of fracturing in the pillar it is necessary to reduce the contact tensile strength to a value of the order of 0.1 MPa. It is not easy to relate this particular value to material properties measured in the laboratory, although it is important to realize that it might not be unreasonable in the context of this type of analysis.

To put the effect of tensile strength of the contacts or bonds in a more physically meaningful context than simply applying a low tensile strength to all contacts it is better to take an alternative approach. In practice, the rock mass that will form the pillar is likely to be permeated by joints and fractures that have no tensile strength. It may be better to attempt to represent such jointing and fracturing explicitly in the UDEC model. This can be achieved by setting the tensile strength of some of the contacts in the model but not all to zero.

After some consideration it was decided not to explicitly set specific joints or contacts to have zero tensile strength as this would require some considerable manual intervention and would cause problems with repeatability across different block assemblages and pillar dimensions. It was decided to adopt an approach whereby a percentage of the contacts in the block assembly would be assigned a zero tensile strength. This was achieved by initially setting all contacts in the model to have a finite tensile strength. FISH was then used to scan through all contacts in the model and set a pre-specified proportion of them to have a zero tensile strength. The resulting contacts with zero tensile strength can be considered as existing fractures in the rock mass.

3.3.7.4 Pre-Existing Rock Fracture

Brady and Brown suggest that rock cannot support tensile stress because there are pre-existing fractures in the material. They advise that a zero tensile strength cutoff should be used for modelling a rock mass in the context of continuum modelling; this may not be appropriate for discontinuum modelling. If a global finite value for a tensile strength cutoff is applied to all contacts in the model then this assumes that there are no pre-existing fractures in the rock mass. A better assumption might be that a certain percentage of the potential fractures in the rock mass are already in existence. It is possible to represent such fracturing in the model by setting a certain percentage of all contacts to have a zero tensile strength cutoff value. This can be achieved by initially assigning all contacts some global tensile strength cutoff value. Subsequent to this contacts can be scanned with a UDEC FISH function and a pre-defined
percentage can have their tensile cutoff strength reassigned as zero. Two methods were tried in this analysis to create pre-existing fractures with a tensile strength of zero.

Figure 3-32: Fracture patterns and deformation-stress curves produced for increasing values of the tensile strength cutoff value: the figure in the central column shows direction of shear (i.e. PLOT DSHEAR).
Initially, all contacts are assigned some finite tensile strength cutoff value; in this case it is 12.5 MPa which is one tenth of the UCS of the rock mass. Subsequent to this, all contacts are scanned in the discontinuum region and every n\textsuperscript{th} contact has its tensile strength cutoff set to zero. This results in single contacts in the mesh with a zero tensile strength surrounded by contacts with a finite tensile strength cutoff. This was achieved in a UDEC FISH function by scanning contacts and simply changing every n\textsuperscript{th} contact value. This results in a multitude of fractures that are one contact in length. It soon became clear that this approach is not extensible: it is not possible to change more than 50% of the contacts as it is only possible to go down to every second contact. This approach was rapidly abandoned and is not reported here.

The second approach involves changing a set of contacts within a given number of contacts. All contacts are changed up to a given count and then the next block of contacts are ignored until some count limit is reached and contact changing begins again. It is expedient to set the count limit to 100 so that the number of contacts to be changed can be expressed as a simple percentage. With this method, a “bunch” of adjacent contacts are set to have zero tensile strength in proportion to a specified percentage. This method allows complete extensibility to specify rock fracturing from zero to one hundred percent. This method was adopted in this analysis. What this means is that there are clumps of contacts with zero tensile strength and clumps of contacts that are in an unfailed tensile state. This represents fractures that are more than one contact in length: this is probably more realistic than the one contact long fractures produced by Method 1. Method 2 was adopted in this analysis to represent existing fractures in the rock mass as it was considered both a better representation of fractures in a true rock mass and it is more controllable in the UDEC simulations. Results from the simulations for fracture densities of 10%, 20%, 50% and 75% indicate that the ultimate strength of the pillar does not vary significantly as a consequence of the fracture density (Figure 3-33). However, the residual strength of the pillar is significantly influenced by the initial fracture density of the rock mass. For a fracture density of 10% the residual strength of the pillar climbs back to its initial ultimate strength of 45MPa with a stiffness that is around half that of the original “unfailed” pillar. As the original rock mass fracture density is increased, the residual strength and stiffness of the pillar is reduced. At a fracture density of 75% the residual strength of the pillar fluctuates between about 15 MPa and 20 MPa. It is not clear how realistic the yield strengths produced in this set of simulations are. An ultimate pillar strength of between 45 MPa and 50 MPa seems unrealistically low but it is impossible to say with any certainty without underground measurements of the yield strength of “crush” pillars. Although the fracture density does not have any significant affect on the yield strength of the pillar the UCS of the intact rock mass does effect the ultimate strength of the pillar as is discussed below.
3.3.7.5 Uniaxial Compressive Strength

The uniaxial compressive strength (UCS) of a purely Mohr-Coulomb material is given by Goodman (1989) as

$$q_u = 2S_i \tan \left(45 + \frac{\phi}{2}\right)$$

where $q_u$ is the UCS, $S_i$ is the cohesive strength and $\phi$ is the angle of internal friction of the rock mass. It is possible to predict UCS for various values of cohesive strength and angle of internal friction (Table 3-3) using this equation. It is also possible to model a UCS test using UDEC with specific values for the cohesive strength and the slip friction angle. Such UCS models can be undertaken on the type of block assembly used in the analyses discussed in this report. Such an analysis has been partly completed by Lightfoot (2006) and it is clear that there is not a direct relationship between the predicted UCS and the modelled UCS – Table 3-4..
Table 3-3: Predicted UCS of a rock mass from Goodman’s equation given specific values for the friction angle and the cohesive strength of the rock mass (Lightfoot, 2006).

<table>
<thead>
<tr>
<th>Cohesion (MPa)</th>
<th>10</th>
<th>20</th>
<th>30</th>
<th>40</th>
<th>50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Internal Friction</td>
<td>20</td>
<td>40</td>
<td>60</td>
<td>80</td>
<td>100</td>
</tr>
<tr>
<td>0</td>
<td>24</td>
<td>48</td>
<td>72</td>
<td>95</td>
<td>119</td>
</tr>
<tr>
<td>10</td>
<td>29</td>
<td>57</td>
<td>86</td>
<td>114</td>
<td>143</td>
</tr>
<tr>
<td>20</td>
<td>35</td>
<td>69</td>
<td>104</td>
<td>139</td>
<td>173</td>
</tr>
<tr>
<td>30</td>
<td>43</td>
<td>86</td>
<td>129</td>
<td>172</td>
<td>214</td>
</tr>
<tr>
<td>40</td>
<td>55</td>
<td>110</td>
<td>165</td>
<td>220</td>
<td>275</td>
</tr>
</tbody>
</table>

Table 3-4: Modelled UCS of the rock mass derived from UDEC simulations with a given slip friction angle and cohesive strength for the rock joints (Lightfoot, 2006).

<table>
<thead>
<tr>
<th>Cohesion (MPa)</th>
<th>10</th>
<th>20</th>
<th>30</th>
<th>40</th>
<th>50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slip Friction</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>10</td>
<td>53</td>
<td>105</td>
<td>157</td>
<td>210</td>
<td>262</td>
</tr>
<tr>
<td>20</td>
<td>57</td>
<td>113</td>
<td>170</td>
<td>228</td>
<td>284</td>
</tr>
<tr>
<td>30</td>
<td>63</td>
<td>125</td>
<td>187</td>
<td>249</td>
<td>312</td>
</tr>
<tr>
<td>40</td>
<td>71</td>
<td>142</td>
<td>212</td>
<td>284</td>
<td>354</td>
</tr>
<tr>
<td>50</td>
<td>87</td>
<td>175</td>
<td>262</td>
<td>350</td>
<td>400</td>
</tr>
</tbody>
</table>

Lightfoot used a “sample” of the same basic mesh as used in the analyses of crush pillars described here. The sample was made to be twice as high as it is wide and it was subjected to controlled displacement loading from the top and bottom by means of modeled solid platens. The friction angle between the rock sample and platens was set to a realistic value of twelve degrees. The sample was compressed and UDEC histories of stress and displacement were recorded in the loading platens. From these histories it was possible to plot deformation-stress curves for the sample. The simulations were performed for various contact slip friction angles.
and cohesive strengths. The yield stress for each test was simply read-off from the corresponding deformation-stress curve produced during the simulation.

It was found, during the course of these numerical tests, that the value of the tensile cutoff had very little influence on the UCS providing it had some finite value. If the tensile strength of the sample was set to zero it simply “unraveled” as soon as the displacement loading was applied.

There are at least two possible root causes of the discrepancy between modeled and predicted UCS. The first problem lies with the geometry of the test specimen: in practice, the sample is a small cylinder of rock, while in the UDEC simulation it is an infinitely long, parallel sided “brick”. It is clear that this discrepancy between sample geometry of the real specimen and the UDEC simulation would result in UDEC over estimating the UCS of the test sample. The second problem lies with the very nature of the friction angle determined in the laboratory and that applied in the UDEC simulations. The friction angle determined in the laboratory tests is an abstract material property of the rock continuum more correctly termed the “angle of internal friction”. The friction angle applied to the block contacts in the UDEC simulation is the slip friction angle of the UDEC joint. From laboratory tests performed on samples the joint, or surface, slip friction angle is generally significantly lower than the angle of internal friction obtained from triaxial compressive stress tests for the same rock type.

At this stage, no attempt has been made to analyse the relationship between predicted and modeled UCS of the rock specimen. The modeled values of UCS for various values of slip friction angle and cohesive strength are simply used to assign values to these parameters in the UDEC model based on the laboratory determined UCS of the rock mass. Laboratory determined values of internal friction angle and cohesive strength are not used as a direct basis for the values of friction and cohesion used in the UDEC simulations. Simulations were performed on pillars of the same size and geometry for three different values of the UCS of the intact rock mass. Using the values given in Table 3-4, values for slip friction angle and cohesive strength for intact rock with UCS values of 125 MPa, 212 MPa and 249 MPa were selected (Table 3-5). In all three cases the pillar was set at 75% fractured.

Table 3-5: Rock mass parameters for angle of friction and cohesive strength used to represent three different values of UCS for the pillar.

<table>
<thead>
<tr>
<th>Rock Mass UCS (MPa)</th>
<th>Slip Friction Angle (degrees)</th>
<th>Cohesive Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>125</td>
<td>30</td>
<td>20</td>
</tr>
<tr>
<td>212</td>
<td>40</td>
<td>30</td>
</tr>
<tr>
<td>249</td>
<td>30</td>
<td>40</td>
</tr>
</tbody>
</table>

Simulations were undertaken to assess the ultimate “strength” of each of the crush pillars with the three different rock strengths. The pillars were loaded to 40mm of convergence as described above which was sufficient to drive them beyond their ultimate yield point. Histories of
deformation versus stress were tracked during the simulations to determine the actual ultimate strength of the three pillars (Figure 3-34).

With a 75% fracture density, the ultimate strength of the 125 MPa UCS rock mass is only 45 MPa, and for the pillars with an intact rock mass UCS of 212 Mpa and 249 MPa it is only 85 MPa and 88 MPa respectively. It is difficult to judge how realistic these values are without additional field data, although they do seem to be unusually low. As discussed above, the fracture density of the rock mass does not seem to have much effect on the ultimate strength of the pillar; it only affects the pillars residual strength. It is clear from this analysis that the UCS of the intact rock mass is the controlling factor on the ultimate strength of the pillar.
Intact rock mass UCS of 125MPa

Ultimate pillar strength of 45MPa

Intact rock mass UCS of 212MPa

Ultimate pillar strength of 85MPa

Intact rock mass UCS of 249MPa

Ultimate pillar strength of 88MPa
3.3.8 Fracture Growth

All results presented so far showing fracture patterns in the crush pillar do so at a single snapshot in any one of the given simulations. However, the loading applied in UDEC is progressive and the fractures in the pillar grow progressively with the increased effect of loading. All subsequent runs discussed from this point onwards involve a constant displacement loading rate of 0.05 units (metres in this case) per unit of simulation time.

In the case of UDEC, the deformation of the pillar with progressive simulation time follows a physically meaningful path which is analogous to progressive failure of a real pillar with increasing stope convergence. It is possible to pause the UDEC simulation at any time and observe the state of fracturing in the pillar at that particular point in simulation time.

In the case of a pillar with no intrinsic tensile strength (Figure 3-35) the edges of the pillar begin to slab at a very early stage (less the 5mm of convergence). This is indicative of tensile failure along vertical partings at the edge of the pillar. Some horizontal and inclined shearing is observed in the footwall and hangingwall of the pillar at its edges. No inclined shearing is observed in the pillar itself. Even at this stage it is clear that the behaviour is beginning to adopt an asymmetrical pattern. The tensile slabbing of the pillar edge continues to beyond 10 mm of convergence at which point the asymmetry of fracture development is seen to become even more pronounced.

As convergence of the pillar exceeds 15 mm the overall fracture pattern is seen to alter quite drastically. At some point prior to this the edges of the pillar begin to disintegrate and the slabbing along vertical fractures is seen to progress deeper into the pillar. The asymmetry in the fracture growth becomes extremely pronounced with the left hand side of the pillar continuing to
fail predominantly along vertically oriented fractures. However, the right hand side of the pillar exhibits a significant tendency towards inclined shear fracture and failure along horizontal planes. Within the centre regions of the pillar some discontinuous and vague inclined shear fracturing is observed within the pillar itself. As the convergence reaches 20 mm, the inclined shearing within the pillar becomes more continuous and more distinct. At 25 mm of convergence the outer skin of the left hand side of the pillar exhibits shearing between the tensile fractures and shows disintegration that is beginning to look more similar to the right hand side of the pillar. However, there is a distinct transition from this disintegration behaviour from what appears to be continued tensile slabbing deeper into the pillar at a distance of roughly one quarter the pillar width into the pillar.

The pillar with an intrinsic tensile strength and pre-existing fracturing shows markedly different patterns in fracture growth to that described above for the pillar with no tensile strength. At 5 mm of convergence the pillar edges exhibit some tensile slabbing but it is significantly reduced and less continuous from that of the previous case (Figure 3-36). As convergence continues, shear fracturing of both the edges and, to a much lesser degree, through the centre of the pillar initiates at a much earlier stage than in the previous case. Already at 10 mm convergence a shear band can be seen to have started developing through the centre of the pillar. With continued convergence up to the point it was stopped at 40 mm, the edges of the pillar disintegrate along all orientations of fracturing exhibiting both tensile and shear failure. Again, the fracture pattern is observed to be asymmetrical with more damage appearing on the right hand side of the pillar than on the left hand side.
Figure 3-35: Progressive fracture growth with increasing displacement loading (convergence) for a pillar with no tensile strength.
3.3.9 Material Behaviour and Properties

As a consequence of the development work that was undertaken to this stage and what detailed work was possible to this point it was decided to fix all material properties of the rock mass at standard values for all future simulations unless otherwise stated. The values used for the material properties of the elastic blocks, and the joints during elastic deformation and plastic yield are given in Table 3-6. Obviously, some of the simulations described later in this report involved the varying one or more of these parameters on the overall behaviour of the pillar. When the value of any of these parameters is altered for the sake of such an analysis this is discussed in the text and the actual value that was used is given.

Obviously, the exact material parameters used for each of the specific features of the rock mass model dictate the overall behaviour of the pillar in some way. However, it is not clear in this type of modelling that there is a direct relationship between individual parameters and the overall pillar response to loading. It is clear that there is a reasonably well behaved correlation between slip friction angle and cohesive strength of the rock mass and the simulated UCS of a rock sample. Such a correlation is reflected, to some degree, in the overall behaviour of the pillar described above.

It is not clear what role the tensile strength of the contacts plays in the overall strength of the rock mass and in turn the actual performance of the pillar. This certainly poses issues that are of concern in this type of modelling and clearly points to areas of potential future work if these issues are to be addressed.
Table 3-6: Material properties used for standard pillar simulations described in this report.

<table>
<thead>
<tr>
<th>FEATURE</th>
<th>PARAMETER</th>
<th>VALUE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Block Elastic Properties</td>
<td>Young’s Modulus</td>
<td>59.5 GPa</td>
</tr>
<tr>
<td></td>
<td>Poisson's Ratio</td>
<td>0.19</td>
</tr>
<tr>
<td></td>
<td>Bulk Modulus</td>
<td>32.0 GPa</td>
</tr>
<tr>
<td></td>
<td>Shear Modulus</td>
<td>25.0 GPa</td>
</tr>
<tr>
<td>Joint Elastic Properties</td>
<td>Normal stiffness</td>
<td>200.0 GPa</td>
</tr>
<tr>
<td></td>
<td>Shear stiffness</td>
<td>200.0 GPa</td>
</tr>
<tr>
<td>Joint Plastic Properties</td>
<td>Slip friction angle</td>
<td>30 degrees</td>
</tr>
<tr>
<td></td>
<td>Cohesive strength</td>
<td>20 MPa</td>
</tr>
<tr>
<td></td>
<td>Tensile cut-off strength</td>
<td>0.0 MPa or 12.5 MPa</td>
</tr>
</tbody>
</table>

In addition to problems relating to the overall effects of the tensile strength of the contacts, a further point of concern, at this stage of the work, is the overall stiffness of the pillar while it remains in the elastic region of deformation. The individual blocks have a stiffness of approximately 60 GPa in terms of the elastic (Young’s) modulus. However, overall, the pillar displays a significantly lower elastic modulus. Consider just the single case of a pillar composed of a standard rock mass with 75% pre-fracturing and points of reference one metre directly above and below the centre point of the pillar. From the displacement-stress profile of one of these points (Figure 3-37) it is clear that the slope of the elastic portion for the pillar deformation produces a rise in stress from about 25 MPa to about 43 MPa at the point of pillar failure. This is accompanied by an overall vertical deformation of around 4 mm. Hence, for a stress rise of 18 MPa there is an associated vertical hangingwall displacement of 4 mm, or a total pillar convergence of 8 mm. The distance across the points of reference for the pillar is 2 metres giving a total strain of the pillar of 8.0e-3 / 2.0 or 4 millistrains. This, in turn, indicates an overall pillar modulus of 4.5 GPa which is significantly lower than the elastic modulus of the individual blocks which, as described above, is set at 60 GPa. A probable explanation is the finite normal (and shear) stiffness, which make each individual triangular block more compliant than the true rock mass. The overall effect is not likely to significantly affect the magnitudes of stress (including peak pillar strengths) but will magnify the modeled deformation values.
Figure 3-37: Displacement-Stress curve for a standard pillar with 75% pre-fracturing illustrating the gradient of the elastic portion of the curve which is about 4.5 GPa.
3.4 Determining PFI with UDEC

Trials were undertaken with two methods of directly relating the UDEC results to the PFI measurements. The first method involved analyzing the strains between adjacent grid points within blocks, and the second involved directly analyzing the opening of contacts between blocks. Both methods involved defining a line through the UDEC model that is analogous to the borehole in the field studies. As the actual borehole is drilled horizontally into the middle of the pillar, a horizontal line is used in UDEC positioned halfway up the pillar (Figure 3-38). A “scan” was then undertaken along this line in the model to find all grid points and contacts that either lay directly on the line or were very close to it.

![Figure 3-38: The FISH function scan line as implemented in UDEC: the solid line along the centre line of the pillar represents the borehole itself and the dashed lines above and below represent the zone around the hole which is scanned (this distance is exaggerated in the figure for the sake of clarity).](image)

Both the grid point strain and contact opening methods for determining PFI involve the use of FISH (see Appendix 1) to scan through the appropriate internal UDEC data arrays to find grid points or contacts that lie close to the pre-defined scan line. This scan is undertaken once at the start of the UDEC simulation and a list of grid points and contacts that lie sufficiently close to the scan line is created and stored in memory.
When first attempting to ascertain PFI from UDEC these lists were used to create UDEC History commands with the intention of allowing UDEC to simply track their state. Unfortunately, this approach did not work as UDEC is limited to a total maximum number of five hundred user-specified histories. In general, in the simulations undertaken here, there are far more than five hundred grid points or contacts in the region of interest. It became necessary to use FISH to keep track of specific grid point and contact histories by storing them in memory in specially declared FISH data arrays. At the end of a simulation special purpose FISH functions are used to copy the data recorded in the arrays to ASCII text files that can then be accessed with applications such as MS Excel for further analysis.

3.4.1 Contact Separation

It was decided at a relatively early stage that a method of determining PFI based on the separation of contacts between blocks held the most promise of the two schemes considered. This involves running a FISH scan through the contact data array looking for all contacts that are within a tolerance limit of the scan line. A number of variables were stored for each contact at given intervals during cycling. The variables recorded in this manner were

- normal displacement
- shear displacement
- normal force
- shear force
- x-component of unit normal
- y-component of unit normal

Unfortunately, a number of unforeseen problems arose while developing the method of contact determined PFI. These problems were mostly related to the following issues

- the effect of horizontal contacts
- the presence of spurious false contacts
- automatic contact deletion
- automatic contact creation

In order to understand the nature of these problems and how they were resolved it is necessary to understand what contacts are in UDEC and specifically how they behave in the context of the mesh used in the analyses described here. First, it is necessary to take a closer look at how joints intersect in the block assemblages used in the analyses and then to look at specific issues pertaining to the contacts that are created.

3.4.2 Joint Intersections

Careful inspection of the UDEC discontinuum mesh indicates that there are two, and only two, distinct types of crack intersections that appear routinely throughout the model (Figure 3-39). The first of these intersection types involves the meeting of four cracks, two horizontal and two vertical, at a point (Type 1) and the second involves the meeting of eight cracks (an additional four dipping cracks) at a single point (Type 2).
An alternative approach would be to eliminate the Type 1 intersections. This can be achieved by inserting two additional joint sets in the model as shown by the dashed lines in Figure 3-39. This will, of course, result in a new type of four joint intersection (Type 3) where four dipping fractures meet.

![Figure 3-39: Two types of block-intersection produced in the model block assemblage. The two dashed lines represent one trace each of the two joint sets that would be required to eliminate the Type 1 intersections within the model.](image)

3.4.2.1 Contacts in the Block Assembly

In the case of two adjacent blocks in the current block assembly there are always two contacts between the blocks (Figure 3-40 A). This is true whether the joints between the blocks are diagonal, horizontal or vertical. In the case where four blocks touch to form a Type 1 joint intersection there are always a total of eight contacts between the blocks and four contacts at the Type 1 intersection itself (Figure 3-40 B). At a Type 1 intersection, all joints are either horizontal or vertical. In the case of a Type 2 joint intersection eight blocks are involved and there are a total of sixteen contacts between the blocks (Figure 3-40 C). At the Type 2
intersection itself, there should always be only eight contacts. Horizontal contacts at both Type 1 and 2 joint intersections caused problems for the PFI analysis. In addition to this, false contacts at the Type 2 intersections also gave problems until they could be eliminated from the analysis.

![Diagram showing contacts between blocks in UDEC](image)

**Figure 3-40:** Contacts between blocks in UDEC represented as thick lines. A) Two contacts at either end of a joint between two adjacent blocks. B) Eight contacts between four adjacent blocks and the four central contacts associated with a Type 1 intersection. C) Sixteen contacts between the eight blocks that comprise the “Unit Cell” of the model with the central eight contacts associated with a Type 2 intersection.

### 3.4.2.2 Horizontal Contacts

There are a great many horizontal contacts in the block assemblage (Figure 3-41) and these may well lie within the region of the scan line. In practice, horizontal fractures would not be easily seen in a horizontal borehole and are likely to produce unwanted results in the model. All contacts with an absolute value of the x-component of the unit-normal vector of less than 0.5 are considered to be horizontal fractures and are eliminated from the analysis.
3.4.2.3 False Contacts

During the course of this work, it was observed that spurious contacts are formed in the block assemblage between blocks that are not actually in contact: examples of two such contacts in the current block assemblage are the contacts numbered 1738109 and 3596344 (Figure 3-42). It would appear that, under normal circumstances, these contacts never carry loads: this means that an analysis identifying fractures based on zero normal loads would always, erroneously, identify these contacts as pillar fractures. However, the absence of a contact force on these contacts makes it is easy to identify them at the very early stages of cycling and then be able to eliminate them from future analysis.

Figure 3-41: Extremely close up view of Type 1 block-intersection of four joints showing four contacts, two of which are horizontal and deemed “uninteresting”: the block curvature is a result of the UDEC rounding length.
3.4.2.4 Automatic Contact Deletion

When a UDEC simulation begins, contacts are set up between adjacent blocks as described above. During the course of the simulation any two blocks sharing contacts between each other may move relative to each other. If the two blocks move apart then physical contact between the two blocks may be broken and the contact becomes redundant. Alternatively, if the two blocks slide relative to each other then the location of the two points of the contact on each block will shear apart and the contact may become unhelpful. UDEC’s default behaviour is to delete redundant and unhelpful contacts.

Contact deletion has the unwanted effect of potentially deleting contacts in the list that is used to monitor PFI in the current analysis. This results in the possibility of contacts simply vanishing from the PFI contact list. Fortunately a simple instruction switch in UDEC suppresses contact deletion. However, suppressing contact deletion does have additional consequences as it alters the solution path followed by UDEC and produces a different result (Figure 3-43). In the case of the PFI analysis discussed here contact deletion is suppressed in all UDEC simulations subsequent to those covered in the Model Development section.

Figure 3-42: Extremely close up view of Type 2 block-intersection of eight joints showing two spurious “false contacts”.

3.4.2.4 Automatic Contact Deletion
3.4.2.5 Automatic Contact Creation

In addition to the automatic contact deletion, described above, that occurs in UDEC there is also the issue of automatic contact creation. When two blocks that were not initially in contact come together during the course of a simulation a contact must be created between the two blocks. Also, when two touching blocks slide sufficiently relative to each other it is necessary to form new contacts to ensure that the block interaction is handled correctly. This means that new contacts are forming all the time during the course of a simulation. However, the contact scan occurs only at the beginning of the simulation so any new blocks that may form in the vicinity of the scan line (or borehole) are not accounted for in the simulation. New FISH code was developed to trap new contact events and log the address of the newly formed contact. The new contact address is placed on a temporary list of new contacts. At the next cycle iteration the new contact list is processed by another FISH function to check if any of the new contacts lie along the scan line. If any new contacts are found to lie on the scan line they are appended to the main scan line contact list so that they can be processed as part of the PFI analysis. At this point the temporary contact list is cleared so that the process can be repeated for the next cycle increment. This ensures that all contacts relevant to the PFI analysis are routinely monitored.
3.4.2.6 Input to UDEC

The problem definition for each of the simulations is input into UDEC by means of an ASCII text data file. The data file structure, or format, for each of the simulations described in the pillar strength and PFI determination sections of this report is identical: only the actual parameter values vary with the various simulations. The data file structure and purpose is given in Appendix 1.

3.4.2.7 Output from UDEC

The current implementation for PFI analysis writes seven separate ASCII text output files as given in (Table 3-7). The files contain space-delimited lists of records containing data of contact or grid point variables as they change during the course of the simulation. Each line in the file represents a record of histories for a variable associated with an individual contact. In the case of the six contact output history files, the first column in each record is the UDEC address of the contact, the second column is the cycle number at which the contact was created - non-zero for any contacts that were automatically created during the course of the simulation, the third and fourth columns are the original x and y coordinates of the contact respectively and the remaining columns represent the values of the variable at each history increment.

Table 3-7: Output ASCII text files produced by the PFI analysis FISH functions for post-processing in an application other than UDEC.

<table>
<thead>
<tr>
<th>cNDisp</th>
<th>Contact normal displacements</th>
</tr>
</thead>
<tbody>
<tr>
<td>cSDisp</td>
<td>Contact shear displacements</td>
</tr>
<tr>
<td>cNForce</td>
<td>Contact normal forces</td>
</tr>
<tr>
<td>cSForce</td>
<td>Contact shear forces</td>
</tr>
<tr>
<td>cX</td>
<td>Contact unit normal in the x-direction</td>
</tr>
<tr>
<td>cY</td>
<td>Contact unit normal in the y-direction</td>
</tr>
<tr>
<td>GridPoint</td>
<td>Grid point x-displacements</td>
</tr>
</tbody>
</table>

3.4.2.8 Post Processing Methodology

The results from the ASCII text files created by the FISH code during the UDEC simulations is imported into Microsoft Excel as space delimited text files for post-processing analysis. The following methodology is then implemented in MS Excel to produce a Fracture History
Worksheet (Figure 3-44) and subsequent Fracture Flag Plot (Figure 3-45). The fracture Flag is then used to compute the PFI value for the pillar using the PFI algorithm described below.

Open contact Unit Normal X vectors file: CX.OUT using the space delimited option.

Open contact normal force file: cnForce.OUT using the space delimited option.

Open contact normal displacement file: cnDisp.OUT using the space delimited option.

Merge cnForce and cnDisp workbooks into one workbook.

In CX.OUT workbook move all new contacts first actual unit normal values to column 5.

Insert a new column 5 on sheets cnForce and cnDisp.

Copy CX.OUT workbook column 5 to columns 5 on cnForce and cnDisp sheets.

For all new contacts, ensure that column 6 on sheets cnForce and cnDisp are not zero: assign a value of 1.

On sheets cnForce and cnDisp delete all rows in which the absolute value of column 5 is less than 0.5: these are the horizontal contacts.

On sheets cnForce and cnDisp delete all rows in which the value of column 6 is zero: these are the false contacts.

On sheets cnForce and cnDisp insert a row above row 1 and apply a header line. Address Cycle X Y Unx 1 2 3 4 5 6 7 8 9 10 Sum Flag

Sort all values on sheets cnForce and cnDisp in ascending order on column 3, the x values.

On sheets cnForce and cnDisp delete unwanted steps: save only every 10th step.

On sheet cnForce convert zero normal forces to unit values i.e. 1 = open fracture and non-zero values to 0. Now 1 designates an open fracture and 0 designates a closed fracture.

Sum fracture values of all steps in column P.

If last step has a fracture value of 0 (column O) but the sum (column P) is non-zero then the fracture has been open but it has now closed.

Use the worksheet formula “=IF(P2=0,0,IF(P2+O2=P2,-1,1))” in row 2 of column Q to Set

Open fracture = 1

No fracture = 0

Closed fracture = -1

Copy this function to all necessary rows in column Q.

Graph values in column Q against the x value in column C.
Figure 3-44: A typical Microsoft Excel Fracture History Worksheet with rows representing individual contacts and columns being attributes of the contacts. In this case 10 histories are shown and for each history a 1 represents an open contact and a 0 represents a closed contact. Only a small number of contacts are shown in the figure.

Figure 3-45: A Fracture “Flag” plot for a horizontal line across the centre-line of the pillar: +1 represents currently open contacts, -1 represents contacts that have been open in the past but are now closed and 0 represents contacts that have never opened.
3.4.2.9 The PFI Algorithm in MS Excel

Once the spread sheet of contact flags has been created by the methodology described above it is necessary to compute the PFI of the pillar. A simple algorithm has been developed in MS Excel VBA (Visual Basic for Applications) to accomplish this (Appendix 2).

3.4.3 PFI Results

Fracture Flag Plots were produced for the pillars with three different intact rock strengths described above. PFI values for each of the pillars was then computed which gave values ranging from 38% for the 125MPa UCS pillar (Figure 3-46) up to 59% for the 212MPa UCS pillar (Figure 3-47). Interestingly, the pillar with an intact rock mass UCS of 249MPa gave a slightly lower PFI of 53% (Figure 3-48) than the “weaker” pillar with intact rock strength of 212MPa.

It is clear from the Fracture History worksheets and from the Fracture Flag plots shown above that many of the fractures that have opened up during the course of the simulation actually close up again at a later point in time. This suggests that it is possible for once open fractures observed in the field in real pillars to disappear with time as they are forced closed again.

The fact that the pillar composed of the material with the highest intact rock UCS has a lower PFI than the pillar with a slightly lower UCS may seem unusual but close scrutiny of the behaviour of these two pillars indicates that the “weaker” pillar is deforming in a relatively stable manner with low velocity vectors throughout the mesh (Figure 3-49), whereas, the “stronger” pillar exhibits some degree of unstable disintegration in its lower left hand corner (Figure 3-50). Ejection velocities approaching 3m/s at this location may indicate that this pillar is actually “bursting”.

Figure 3-46: Fracture pattern (with an extended horizontal aspect ratio) and Fracture Flag Plot for the pillar with a UCS of 125 MPa: in this case the PFI is 38%.
Figure 3-47: Fracture pattern (with an extended horizontal aspect ratio) and Fracture Flag Plot for the pillar with a UCS of 212 MPa: in this case the PFI is 59%.
Figure 3-48: Fracture pattern (with an extended horizontal aspect ratio) and Fracture Flag Plot for the pillar with a UCS of 249 MPa: in this case the PFI is 53%.
**Figure 3-49:** Velocity vectors for the pillar with a UCS of 212 MPa with a maximum value of 1.235 m/s internally within the pillar.

**Figure 3-50:** Velocity vectors for the pillar with a UCS of 249 MPa with a maximum value of 2.71 m/s in the left hand footwall of the pillar edge. This pillar may well be “bursting”.

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3.5 Discussion of numerical models

The modelling of crush pillars described here began with two critical questions in mind:

- Can fractures in crush pillars disappear with time?
- Can fractures develop in the core of the crush pillar?

It is arguable that the work described in this report has answered these questions to a satisfactory degree. During the life cycle of a crush pillar fractures do indeed form and open that at a later stage are forced closed by additional dilation of other existing or new fractures. Such closing of fractures would be observed as a disappearance of the fracture in practice. Furthermore, it is clear that pillar fracturing can be pervasive and can, indeed, propagate through the core of the pillar although it initiates at the pillar edges.

The work went further than simply addressing these key questions in attempting to develop a quantitative methodology to compare the results of the models with underground observations. In this endeavour, a full set of tools and a well defined methodology for modelling fracture development has been developed and has been presented in this report. However, the actual modelling of crush pillar behaviour remains in its infancy: much still remains to be achieved. For example it remains necessary to address such issues as:

- Asymmetric mining of pillars
- Pillar geometry in terms of both width and height
- Pillar composition such as different layers or inhomogeneities
- Parting planes within the pillar both in terms of bedding and jointing and faulting

The underground characteristics of these different conditions have been determined and will be better understood with numerical analysis. The approach is novel in terms of modelling brittle rock mass behaviour in the specific context of the overall behaviour of hard rock crush pillars. No claim is made that the work undertaken to date is exhaustive, in fact, it is clear that the work is in its infancy. However, the approach shows considerable promise. At this point the two major points of concern are:

- The effect of contact tensile cutoff strength on the overall pillar strength and behaviour
- The low stiffness of the pillar as a whole

and will require addressing if the approach is to be of any real value in understanding the macroscopic behaviour of a brittle rock mass in a practical environment. In addition to this there are many issues that need to be addressed in terms of the more microscopic aspects of the model. The work described here covers a blocky rock mass cut through by vertical and horizontal joints and conjugate joints dipping at seventy degrees. A very preliminary investigation was undertaken into the necessary block density and size for this type of analysis. Further analysis must include the effect of such factors as:

- Joint sets at other orientations
- The inclusion of more joint sets and, by implication more blocks,
- The effects of block size
- The effect of joint normal and shear stiffness
The potential benefits of the discontinuum modelling approach over more traditional continuum modelling approaches is the elimination of the need for a complex, predefined constitutive law to describe the post peak plastic behaviour of the rock mass. However, if the issues relating to contact tensile strength and system stiffness cannot be satisfactorily resolved then any such benefits would be negated by the need to artificially account for these factors. Although the discontinuum approach to modelling of pillars described in this report has revealed at least two significant problems with the approach, the very potential of the approach illustrated here makes it worth the effort to investigate these issues further in an attempt to develop a method of modelling fractured rock. Such a method may produce significant insights into how fractured hard rock behaves in specific crush pillar situations. In addition, the development of such a model could be extended to other situations where hard rock fracture is of predominant importance, such as hangingwall and sidewall support, face bursting and preconditioning and rock bursting in general, to name just a few.

3.6 Conclusions from numerical modelling

These numerical experiments began with the desire to answer two basic questions. The first of these being "can fractures in a crush pillar disappear with time?". The second basic question was “can the core of a crush pillar fracture?”. The work is at a very early stage but it is possible to derive a number of conclusions at this point. It is also possible to discern a potential way forward from the current status. To date, some of the numerical experiments have shown that blocks that were separate in earlier stages of the model can “clump” and begin to behave as combined blocks as the simulation progresses. This would indicate that fractures in the pillar can “disappear” or close-up with time. Under certain circumstances the core of the pillar can fail: exactly what these circumstances are is unclear at this stage.

A set of tools and a methodology for simulating the quantitative measurement of PFI within a UDEC model has been developed. At this stage, no attempt has been made to add a gravity body force to the system which may induce spalling at the edges of the pillars. The discontinuum models are able to exhibit yield behaviour, true crush behaviour and explosive behaviour without the need to specify complex, macroscopic, plasticity behaviour for the pillar.

The effect of the contact cutoff tensile strength on the behaviour of the rock mass and in turn on the overall behaviour of the pillar is not well understood. The discontinuum pillar model exhibits a system stiffness that appears to be significantly lower than anticipated: the reasons for this are not understood. The 17 000 block models seem to offer a reasonable compromise between acceptable run-times and necessary degrees of freedom. However, to take this work further, it may be necessary to accept the run-time penalties of models with more than 50 000 blocks. This work has only begun to explore the potential of the discontinuum approach to modelling crush pillars and much work remains to be done. The results of the current models need to be studied in more detail. Finally, has this modelling achieved what it set out to do? It has moved in a positive direction down the path on which it set out. At this stage it has probably introduced more questions than it has answered but it must be remembered that the approach is new.

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4 Design charts for crush pillar performance

Many attempts have been made to develop design charts for different pillar types including crush pillars. These are however not universally applicable and often require special adaptations or modifications for each site or even each pillar. This reflects the complexity of the situation. During the course of the detailed monitoring and subsequent data analysis it was found that the multitude of geological environments, the wide range of depths, many different mining conditions and the variety of pillar shapes makes it impractical to develop simple guidelines and design charts.

Rather it is suggested that the PFI is used to measure the pillar condition and thus determine if the crush pillars are behaving correctly in that particular complex combination of variables. It has been shown that the PFI takes into account all variables as it is simply a direct measure of the pillar response, rather than an attempt to calibrate the many different factors.

Section 2 shows that the PFI of different pillar types can be determined. This allows the recognition of correctly designed crush pillars – that is pillars that are actually crushing. As pointed out in Section 1, many crush pillars are not correctly mined and hence need to be checked using this simple technique. The change in PFI with time shows the kind of geotechnical environment in which these pillars are present. These values should be compared with the geotechnical characteristics measured for the underground site to determine if the crush pillars are behaving correctly. If not, the design or implementation of the pillars needs to be re-evaluated.

Two tables, derived from Section 2, indicate the variability in PFIs for different mining and geological environments. These indicate not only the variety of mining conditions but also the variety of geological and geotechnical environments under which crush pillars might be used. It is important to note that the PFI concept can be applied across all of these different conditions and environments. Detailed calibration can be done at a specific site, to see how various pillars within a uniform mining / geotechnical zone are behaving, allowing for further refinement of the pillar design. It also provides a quantifiable way to evaluate and enforce pillar cutting discipline.
Table 4-1: PFI values for different mining conditions.

<table>
<thead>
<tr>
<th>Tributary area</th>
<th>In panel % extraction</th>
<th>Load (MPa)</th>
<th>PFI</th>
</tr>
</thead>
<tbody>
<tr>
<td>66.0</td>
<td>98%</td>
<td>2886.8</td>
<td></td>
</tr>
<tr>
<td>46.0</td>
<td>98%</td>
<td>2794.5</td>
<td>29</td>
</tr>
<tr>
<td>46.0</td>
<td>98%</td>
<td>2297.7</td>
<td></td>
</tr>
<tr>
<td>49.5</td>
<td>98%</td>
<td>2739.8</td>
<td></td>
</tr>
<tr>
<td>48.0</td>
<td>98%</td>
<td>1231.2</td>
<td>27</td>
</tr>
<tr>
<td>8.4</td>
<td>88%</td>
<td>213.2</td>
<td></td>
</tr>
<tr>
<td>46.6</td>
<td>98%</td>
<td>1200.3</td>
<td>16</td>
</tr>
<tr>
<td>76.1</td>
<td>99%</td>
<td>2115.4</td>
<td></td>
</tr>
<tr>
<td>12.3</td>
<td>92%</td>
<td>1051.8</td>
<td>46</td>
</tr>
<tr>
<td>12.3</td>
<td>92%</td>
<td>1124.6</td>
<td>52</td>
</tr>
<tr>
<td>7.9</td>
<td>87%</td>
<td>245.1</td>
<td>62</td>
</tr>
<tr>
<td>29.4</td>
<td>97%</td>
<td>969.8</td>
<td>43</td>
</tr>
<tr>
<td>11.6</td>
<td>91%</td>
<td>453.4</td>
<td>79</td>
</tr>
<tr>
<td>13.9</td>
<td>93%</td>
<td>501.6</td>
<td>45</td>
</tr>
<tr>
<td>37.3</td>
<td>97%</td>
<td>1388.8</td>
<td>57</td>
</tr>
<tr>
<td>9.9</td>
<td>90%</td>
<td>296.9</td>
<td>36</td>
</tr>
<tr>
<td>7.7</td>
<td>87%</td>
<td>196.4</td>
<td>60</td>
</tr>
<tr>
<td>6.3</td>
<td>84%</td>
<td>56.3</td>
<td>95</td>
</tr>
</tbody>
</table>
Table 4-2: Variations in PFI versus stratigraphy to predict pillar behaviour.

<table>
<thead>
<tr>
<th>Stratigraphy</th>
<th>PFI</th>
<th>Hangingwall behaviour</th>
<th>Pillar behaviour</th>
<th>Footwall behaviour</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low cohesion TRC</td>
<td>Much higher than average</td>
<td>Less fractured</td>
<td>Increase in steeply dipping fractures and dilation</td>
<td>Less fractured</td>
</tr>
<tr>
<td>Low cohesion BRC</td>
<td>Much higher than average</td>
<td>Less fractured</td>
<td>Increase in steeply dipping fractures and dilation</td>
<td>Less fractured</td>
</tr>
<tr>
<td>Low cohesion layer in pillar</td>
<td>Average</td>
<td>Less fractured</td>
<td>Dilation along parting with oblique dipping fractures</td>
<td>Less fractured</td>
</tr>
<tr>
<td>Weak hangingwall</td>
<td>Average</td>
<td>More fractured, fall of grounds</td>
<td>Width to height ratio decreases</td>
<td>Less fractured</td>
</tr>
<tr>
<td>Weak footwall</td>
<td>Average</td>
<td>Less fractured</td>
<td>Width to height ratio decreases</td>
<td>More fractured or fractures open up causing footwall heave</td>
</tr>
<tr>
<td>Strong TRC</td>
<td>Lower than average</td>
<td>Steep fractures penetrate into hangingwall</td>
<td>Less fractured</td>
<td>Steep fractures penetrate into footwall?</td>
</tr>
<tr>
<td>Strong BRC</td>
<td>Lower than average</td>
<td>Steep fractures penetrate into hangingwall</td>
<td>Less fractured</td>
<td>Steep fractures penetrate into footwall?</td>
</tr>
</tbody>
</table>
5 Summary and conclusions

An initial survey on the application of crush pillars, followed by detailed underground and numerical modelling work, has produced many results, some of which are unexpected.

From the survey of current crush pillar designs and use it was concluded that the optimal design of pillar layouts requires an improved understanding of the mechanistic behaviour of pillar material, and of the interaction between pillars and the surrounding strata. Many gold and platinum mines use crush pillars with strike dimensions of 2, 3, 4, 5 or 6 m and 2 or 3 m on dip, separated by 0.5 to 3 m wide ventilation holings. These pillars are normally located on strike and with a 1 m siding on the down dip side of the strike gully. The mechanisms involved in pillar behaviour are complex and generalised formulations may not be applicable to all cases. Many factors can affect pillar performance in different environments, such as cohesion, internal friction, contact friction, discontinuities, stress state, panel dimensions and extraction ratio.

There have been a number of studies to estimate the impact of these factors. However, to take all these factors into account in design is time-consuming and expensive. As a result the pillars may not be ideally situated for the particular set of conditions that occur at a specific site. In addition to this, there is often a serious lack of discipline of cutting correct pillar sizes. This can, in principle and also in practice, result in pillar bursting problems. It was therefore recommended that the sizes of crush pillars should be regularly checked and reported to Rock Engineering Departments for necessary remedial action. It was also concluded that it is important to understand the rock mass conditions and responses as accurately as possible.

To do this, the rock mass and mining conditions of forty different sites at nine different shafts have been quantified. In addition, detailed geotechnical monitoring has taken place at eleven sites. Data from closure-ride stations, extensometers, borehole camera surveys, geological mapping, geotechnical mapping and seismic monitoring has been collected.

The response of crush pillars to seismic events (that is their dynamic behaviour) showed that firstly, the fracturing of the pillar occurs continues to occur well behind the face and secondly, that crush pillars scarcely respond to larger distant seismic events. The influence of geological discontinuities and mining induced fractures on crush pillar behaviour has also been established. It was found that if there are suitably oriented discontinuities, either pre-existing geological features such as joints or mining induced shear fractures that formed before the pillar was cut, much of the stress that should have resulted in the fracturing of the reef can actually be accommodated along these discontinuities. The body of the pillar would thus be prevented from crushing and yielding, resulting in a lower PFI. As the load increased with further mining, these discontinuities become unable to move and yield, resulting in a stress build-up, which could not be relieved through movement of the fractured reef zone. In several cases this resulted in the strain-bursting of crush pillars. In the different stratigraphic and structural domains described above, there does not appear to be a possibility of the pillar crushing to form a blocky rock-mass analogous to a UCS specimen.

Closures of between 30 and 50 mm per week were measured at different sites, from deep level gold mines to shallow platinum mines. Where crush pillars fractured as designed, the closure appeared to be relatively constant across the panels. This indicates that when these pillars are
acting correctly, the hangingwall rock mass acts as a single beam, usually parting on stratigraphic weaknesses. Borehole logging and mapping of pillar holings created an extensive database of fracture characteristics of crush pillars across many different reefs, depths and ages. The fracturing of other pillar types was also quantified to allow the comparison of crush pillar behaviour with these different pillars. From this database the Pillar Fracture Index (PFI) was developed to quantify, especially, crush pillar behaviour. It is defined as

$$PFI = \frac{\sum \text{(lengths of hole with fractures > 10cm apart)}}{\text{Total length of hole}} \times 100$$

The PFI provides a single number that encapsulates all the characteristics of the pillar and thus allows the easy comparison of different pillars. It was found that all crush pillars had Pillar Fracture Indices of less than 45. Slight variations in the PFI can be ascribed to variations in the geotechnical conditions and age of specific types of pillars. For example low PFI values can be due to both abundant geological discontinuities (such as around potholes) or the fact that the area was mined out long ago. PFI values are similar in different mines on different reefs, where the age of the pillars are similar, despite the fact that these pillars were formed on different reef horizons. Pillars that initially have a PFI of a yield pillar, dropped to a crush pillar PFI after several weeks. It appears as though there is a continuum in rock mass behaviour (including fracturing, closure and pillar dilation) between crush and yield pillars, highlighted by the change in PFI values with time. Even poorly cut grid pillars can act as either crush or yield pillars.

From the underground studies, the following conclusions can be drawn on crush pillar behaviour:

1. Crush pillars are not affected by seismicity from distant events. No correlation between the microseismicity (i.e. fracturing) of the pillar and larger distant events was found.
2. Crush pillars are not completely formed ahead of the mining face. Seismic information, detailed mapping and monitoring and PFI evidence show that fracturing continues for tens of metres behind the face.
3. The fracturing of a crush pillar is dependant on the mining history, with fractures developing parallel to the free edge and wrapping around the pillar as each subsequent side is developed.
4. Where low cohesion reef contacts are present, the fracturing of crush pillars is like that of a UCS specimen compressed with frictionless platens (see Figure 2-68).
5. Where weak hangingwall is present, fallout can change the width to height ratio of the crush pillar and thereby cause an increase in fracturing.
6. Where a weak footwall is present, this can slide into gullies allowing the dilation of fractures in the pillar.
The PFI has been found to be an accurate measurement of pillar conditions which also shows:

1. There is a strong relationship between the PFI and pillar load and percentage in-panel extraction. These show that the PFI is an effective measure of the rock mass conditions.

2. There may be a different fracture rate (determined from PFI) for crush pillars in different areas, and also potentially a correlation between the rate of PFI change and closure

The PFI has been proven a useful indicator of pillar conditions, allowing evaluation of the different mining and geotechnical conditions to which pillars are subjected. It also allowed the calibration of the numerical models with actual underground conditions.

Numerical modelling was desirable to better understand crush pillar behaviour under different loading conditions. Preliminary linear elastic MINSIM modelling was carried out to estimate the order of magnitude of pillar loading, from first formation to into the back areas. Results indicating a build-up in stress with time on the pillars were obtained. However, as an elastic solution, MINSIM does not take into account the strain-softening behaviour of the pillars due to failure, hence the pillar stresses increase steadily as the face distance increases. Modelling of in-panel crush pillars as an elastic material thus provides only a very marginal additional insight into the expected levels of ‘peak’ stress in the pillars.

The underground studies of crush pillars as part of this project indicated two interesting and previously unconsidered aspects of behaviour of in-situ crush pillars. The mechanisms behind these observations were investigated using a 2D distinct element / finite difference code (UDEC). The UDEC experimental models began with the desire to answer two basic questions that arose as a result of the underground observations. The first of these was “can fractures in a crush pillar disappear with time?” and the second “can the core of a crush pillar fracture?”. The work is at a very early stage but it is possible to derive a number of conclusions at this point. The numerical experiments have shown that blocks that were separate in earlier stages of the model can “clump” and begin to behave as combined blocks as the simulation progresses. This would indicate that fractures in the pillar can “disappear” with time. Under certain circumstances the core of the pillar can fail: exactly what these circumstances are is unclear at this stage. Numerical modelling has also introduced more questions on the nature of this behaviour, indicating that our knowledge of crush pillars is not yet complete. Encouraging initial results including an extremely strong match-up with modelled and measured PFI values is shown in Figures 5.1 and 5.2.
Figure 5-1: Actual PFI measurements underground – note that when the curve flattens out the PFI is equal to about 25. Compare with Figure 5.2.

Figure 5-2: UDEC model results showing plot of fracture patterns at 40 mm convergence, and deformation-stress curve. The PFI determined from this experiment was 25.

During the course of the detailed monitoring and subsequent data analysis it was found that the multitude of geological environments, the wide range of depths, many different mining conditions and the variety of pillar shapes makes it impractical to develop simple guidelines and design charts. Rather it is suggested that the PFI is used to measure the pillar condition and thus determine if the crush pillars are behaving correctly in that particular complex combination of variables.
To summarise the conclusions, it may be said that:

1. Crush pillars are often poorly cut and not optimally designed in South African hard-rock mines.
2. Crush pillars are not affected by nearby seismic events, micro-seismicity (indicative of brittle fracture) occurs over a range of the pillar’s life.
3. Crush pillars are part of a system that includes the foot and hangingwall and these often have an over-riding impact on pillar conditions. In addition major geological discontinuities, both stratigraphic and structural, control the way in which a pillar crushes and deforms.
4. Although crush pillars appear to have a relatively unfractured, almost solid core when initially cut, they do not fail violently, but continue to gradually fracture inwards for a distance of up to 60 m from the face.
5. PFI is a simple, consistent and accurate measure of the pillar condition and behaviour.
6. PFI quantifies the impact of various geological / geotechnical characteristics on pillars.
7. The influence of different mining conditions can be quantified using the PFI.
8. Crush pillars tend to behave in a common way in similar mining and geotechnical environments, irrespective of what reef is being mined.
9. Crush pillars continue to crush (fail in a brittle manner) well behind the face. This is evidenced by geotechnical, seismic and numerical modelling information.
10. Very detailed UDEC investigations have qualitatively defined pillar behaviour, but these results need to be quantified both by additional modelling and by supplementing the database of underground information with more PFI measurements.
11. The numerical experiments have shown that separate blocks can combine and behave as combined blocks, indicating that fractures in the pillar can “disappear” with time as observed underground.
12. This modelling also confirmed the underground studies that under certain circumstances the core of the pillar can fail at a later stage of the pillar’s life.

To assess the behaviour of different pillars under different mining and geotechnical conditions, the PFI can be measured. This quantification gives a measure of the pillar’s condition relative to the (small) database of pillar performances. The numerical modelling approach outlined here can be used to qualitatively understand the actual environmental conditions for the pillars. It should however be remembered that these models can only become fully quantifiable given sufficient input data (including PFI, closure, pillar dilation and stress measurements) and model experiments.
6 References


7 Appendix 1

At the start of a simulation the UDEC command NEW is used to initialize the UDEC state that is ready for a completely new simulation. A UDEC block is then defined by specifying a rounding length for the block and a block size, in this case from minus 3 metres to plus 3 metres in the x-direction and minus 5 metres to plus 5 metres in the y-direction.

```
NEW
  round 0.001
  block -3,-5 -3,5 3,5 3,-5
```

The original block is then subdivided into three separate blocks to allow the discontinuum zone to be applied to the central region of the UDEC block assemblage while leaving the upper and lower blocks intact.

```
crack -3,-1.6484864 3,-1.6484864
ncrack -3, 1.6484864 3, 1.6484864
```

The central region of the block is further subdivided horizontally to define a potential reef plane through the centre of the block. This allows the reef plane to be cracked separately from the immediate hangingwall and footwall regions should this be required. This was not done in any of the simulations presented here but the commands have been retained for completeness.

```
crack -3,-0.8242432 3,-0.8242432
ncrack -3, 0.8242432 3, 0.8242432
```

This central region of the pillar is then marked as Joint Region 1.

```
jregion id=1 -3,-1.6484864 -3,1.6484864 3,1.6484864 3,-1.6484864
```

The central region is then subdivided into the small triangular blocks by means of four joint set commands. The first two commands create the dipping joints, the third command creates the vertical joint set and the last command creates the horizontal joint sets.

```
jset  70,0 5,0 0,0 0.046984625,0 0,0 range jregion=1
njset 110,0 5,0 0,0 0.046984625,0 0,0 range jregion=1
njset 90,0 5,0 0,0 0.025,0 0,0 range jregion=1
njset 0,0 5,0 0,0 0.068686925,0 0,0 range jregion=1
```

At this point, the whole UDEC block assemblage is meshed into triangular, constant strain finite difference zones and the current state is saved to disk file.

```
gen edge 0.5
save pfi01.mesh.sav
```

The rock mass properties were defined for two different material properties. The first values (mat=1) apply to the intact, or unfractured, joints in the rock mass and the second values (mat=2) are those used for prefractured joints and for new contacts that form between blocks during the course of the simulation.
prop mat=1 density=2700 k=32e9 g=25e9
gprop jmat=1 jkn=200e9 jks=200e9 jfric=30 jcoh=20e6 jten=12.5e6
prop mat=2 density=2700 k=32e9 g=25e9
prop jmat=2 jkn=200e9 jks=200e9 jfric=30 jcoh=20e6 jten=0.0

The joint constitutive model was changed from the UDEC default to a constitutive law (jcons=5) that includes a fracture flag that ensure once a contact has failed in tension or shear it will remain failed for the rest of the simulation. This model was also set as the default model for all new contacts formed during the course of the simulation. The default joint type was set as material type 2 to ensure that any new contacts that form have the material properties of failed contacts. Finally, the material type of all initial joints was reset to be material type 1 that is unfailed contacts.

cha jcons=5
set jcondf=5
set jmatdf=2
change jmat=1

The original UDEC block assembly was loaded with stress boundary and internal (in situ) stress values to simulate a depth of around 1 000 metres.

boundary stress -1.0e6 0.0 -27.0e6
insitu stress -1.0e6 0.0 -27.0e6

The left and right hand side boundaries of the block were defined as axes of symmetry by fixing there x-direction velocities at zero.

boundary corner 230 251 xvelocity=0.0
boundary corner 272 293 xvelocity=0.0

Numerical control parameters were set to specify automatic global damping with mass scaling to ensure efficient numerical damping for the system as a whole. The contact overlap tolerance was set at an arbitrarily high value to prevent UDEC from aborting a simulation if there is a small contact overlap. The default value for maximum acceptable contact overlap is one half of the rounding length which, in this case because of the very small rounding length, would result in UDEC aborting the run at unacceptably low values of contact overlap. Automatic contact deletion is suppressed to ensure that all contacts in the original scan-line list continue to exist throughout the entire course of the simulation.

damp auto
set ovtol=100.0
set delc off

The standard UDEC history increment was set at 100 cycles to reduce the number of history records that are stored by a factor of ten from the default number: the default history cycle increment is ten. The first history was set to track the maximum unbalanced force in the system.
This is not really a particularly useful measure for displacement driven simulations when it is the magnitude of displacement that is of interest rather than out-of-balance force, never-the-less it was tracked for the sake of completeness. Histories of y-direction stress and displacement were set to be tracked at a point in the centre of the block and one metre up into the hangingwall. As displacement histories are recorded at grid point locations and stress histories are tracked at zone centroids the actual histories cannot both be located at exactly this point: UDEC will find the nearest grid point and the nearest zone centroid to the point specified.

\[
\text{his n=100}
\]

\[
\text{history unbal}
\]

\[
\text{history syy(0,1)}
\]

\[
\text{history ydis(0,1)}
\]

\[
\text{save pfi01.init.sav}
\]

The main FISH function definitions and declarations are loaded into UDEC memory by calling the FISH header file FishHead.fis and the file containing the PFI FISH functions PFI.fis.

\[
\text{call FishHead.fis}
\]

\[
\text{call pfi.fis}
\]

Once the PFI FISH functions and parameters have been defined and declared values can be set for them for the current simulation. In this case a vertical elevation (y-level) was set for the scan line and a vertical range within which to scan above and below the scan line was specified. In addition the beginning and end of the scan line was specified in the horizontal direction. Once the scan line parameters were set the functions to execute the contact scan line and the grid point scan line were called.

\[
\text{/*// Define the PFI scan line parameters.}
\]

\[
\text{set yLevel = 0.0}
\]

\[
\text{set yRange = 0.02}
\]

\[
\text{set xStart = -1.49}
\]

\[
\text{set xEnd = 1.49}
\]

\[
\text{ContactScanLine}
\]

\[
\text{GridpointScanLine}
\]

The FISH histories were initialised by setting the cycle increment to 1000 cycles and then loaded into memory with a call to the StartFishHistory function. This function implements the UDEC set fishcall command to ensure that a check is done at each individual UDEC cycle increment to see if the FISH history should be stored. It also implements an alternative form of the same command to store data needed update the scan line contact list any time a new contact is formed. The contact scan line is subsequently updated at the next UDEC cycle increment based on data stored in temporary new-contact data array.

\[
\text{set nHis = 1000}
\]

\[
\text{StartFishHistory}
\]
The intact block assembly is cycled to consolidation to ensure that all contacts are loaded according to the pre-specified boundary conditions. Such a consolidation phase is generally required for all UDEC simulations of this nature.

```
cycle 1000
save pfi01.cons.sav
```

Standard UDEC zone and grid point histories of vertical stress and displacement are requested at a number of points in the hangingwall above the location of the pillar.

```
his syy(-1.5,1.0) ydis(-1.5,1.0)
his syy(-1.0,1.0) ydis(-1.0,1.0)
his syy(-0.5,1.0) ydis(-0.5,1.0)
his syy(-0.0,1.0) ydis(-0.0,1.0)
his syy( 0.5,1.0) ydis( 0.5,1.0)
his syy( 1.0,1.0) ydis( 1.0,1.0)
his syy( 1.5,1.0) ydis( 1.5,1.0)
his syy(-1.5,1.8) ydis(-1.5,1.8)
his syy(-1.0,1.8) ydis(-1.0,1.8)
his syy(-0.5,1.8) ydis(-0.5,1.8)
his syy(-0.0,1.8) ydis(-0.0,1.8)
his syy( 0.5,1.8) ydis( 0.5,1.8)
his syy( 1.0,1.8) ydis( 1.0,1.8)
his syy( 1.5,1.8) ydis( 1.5,1.8)
```

The stope material adjacent to the pillar is deleted to simulate mined reef and leave a pillar with a width of 3 metres.

```
delete -3.0 -1.5 -0.8242432 0.8242432
delete  1.5  3.0 -0.8242432 0.8242432
save pfi01.orig.sav
```

The degree of pre-fracturing of the rock mass is set by first specifying the region in which to change contact material properties. The StartStep parameter is set to specify which contact to start with in the UDEC contact data array. The percentage of contacts to change is set and the material type to which they are to be reset is specified. Finally a call to the ChangeContactEx FISH function is executed to change the requested percentage of contacts in the specified range to a pre-fractured state.

```
set xStart    = -3.0
set xEnd      =  3.0
set yStart    = -1.6484864
set yEnd      =  1.6484864
```
set StartStep = 1
set percentage = 75
set nMat = 2

ChangeContactsEx

The previous boundary conditions were applied to consolidate the block assembly, it is now necessary to apply boundary conditions to load the pillar in terms of simulated mining. Boundary velocity conditions are applied to the top and bottom boundaries of the assemblage to simulate the effect of stope convergence on the pillar. Velocities of 0.05 units (metres) per simulated time step were applied in a downward direction on the upper boundary and in an upward direction on the lower boundary. The rate of convergence is double the velocity applied to the individual boundaries.

boundary corner 251 272 yvelocity = -0.05
boundary corner 293 230 yvelocity = 0.05
save pfi01.s000.sav

The model was cycled in a series of increments for a specified period of simulated time. The simulated time for each increment was set at 0.05 of one simulated second. The number of actual increments and save files specified was dependent on how much final convergence was required. Five increments is equal to a convergence of 25mm on the pillar and eight cycles is equivalent to 40mm of convergence.

cycle time 0.05
save pfi01.s001.sav
cycle time 0.05
save pfi01.s002.sav
cycle time 0.05
save pfi01.s003.sav
cycle time 0.05
save pfi01.s004.sav
cycle time 0.05
save pfi01.s005.sav

On completion of the simulation the FISH file, output.fis, containing the definitions of the FISH functions for writing any required ASCII text files was called. The output functions are executed within a UDEC data file called output.dat.

call output.fis
call output.dat
RETURN

At this point the UDEC simulation concludes and control of the UDEC environment is restored to the user for any interactive analysis of the final state of the simulation.
Appendix 2

The VBA function that implements the algorithm scans the column of fracture flags looking for open cells on the Excel worksheet that indicate contacts that are not open. It then sums the distance between consecutive contacts that are not open in a manner similar to the way in which PFI is computed for the boreholes in practice. The code keeps tab of consecutive lengths that are greater than the specified length, in the case of this analysis that is ten centimeters. From this it is possible to compute the PFI based on the sum total of these “chunks” and the length of the scan line.

```vba
lngColX = 3
lngRow = Selection.Row
lngCol = Selection.Column
sngSum = 0#
sgTotal = 0#
sngCaliper = 0.1
blnSum = False
i = lngRow – 1
Do While (True)
i = i + 1
If (IsEmpty(Cells(i, lngCol))) Then Exit Do
lngVal = CLng(Cells(i, lngCol).Value)
If (lngVal < 1) Then
If (blnSum) Then
sngDeltaX = CSng(Cells(i, lngColX).Value) - CSng(Cells(i - 1, lngColX).Value)
sngSum = sngSum + sngDeltaX
Else
blnSum = True
sngSum = 0#
End If
Else
blnSum = False
End If
If (sngSum > sngCaliper) Then
If (sngTotal = 0#) Then sngTotal = sngCaliper
sngTotal = sngTotal + sngDeltaX
```

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End If
Loop
sngLength = CSng(Cells(i - lngRow, lngColX).Value) - CSng(Cells(lngRow, lngColX).Value)
strMsg = "PFI = " & CStr(Format(((sngTotal / sngLength) * 100#), "##0.00")) & "%"
Call MsgBox(strMsg, vbOkOnly)