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LIST OF SYMBOLS

Stress	σ
Shear stress	τ
Strain	ε
Young's modulus	E
Poisson's ratio	ν
Maximum deflection	$\delta_{\max}$
Maximum fibre stress	CT
Density	ρ
Gravitational acceleration	g
Beam thickness	t
Span	L
Cohesion	Co
Internal friction angle or normal probability density function	ϕ
Plastic shear strain	ε_p
Dilation angle	ψ
Pillar strength	PS
Pillar height	h
Effective pillar height	h_e
Pillar width	w
Pillar effective width	w_e
In situ cube strength	K_i
Pillar width strengthening parameter	α
Pillar height strengthening parameter	β
Pillar length strengthening parameter	a
Pillar width-to-height strengthening parameter	b
Pillar width-to-height ratio	R
Pillar volume	V
Pillar safety factor	SF
Likelihood function	L_i

Standard deviation

S

Normal probability density function

Φ

1 General introduction

This thesis describes the behaviour of the rock mass during the stoping of narrow tabular reefs on South African Bushveld Complex platinum mines, based on underground measurements made at three Merensky Reef sites. The aim of the investigation was to develop a robust design methodology for Merensky Reef 'crush' pillars and to determine the strength and behaviour of these pillars.

1.1 Location of the Bushveld platinum orebodies

The Bushveld Complex is a large layered igneous intrusion, which spans about 350 km from east to west. This region is situated north of the city of Pretoria in the northern part of South Africa (Figure 1-1). This remarkable geological phenomenon hosts not only the majority of the world's platinum group metals but also contains nickel and gold. There are also vast quantities of chromium and vanadium in seams parallel to the platinum ore bodies, some hundreds of metres into the footwall and hangingwall, respectively.

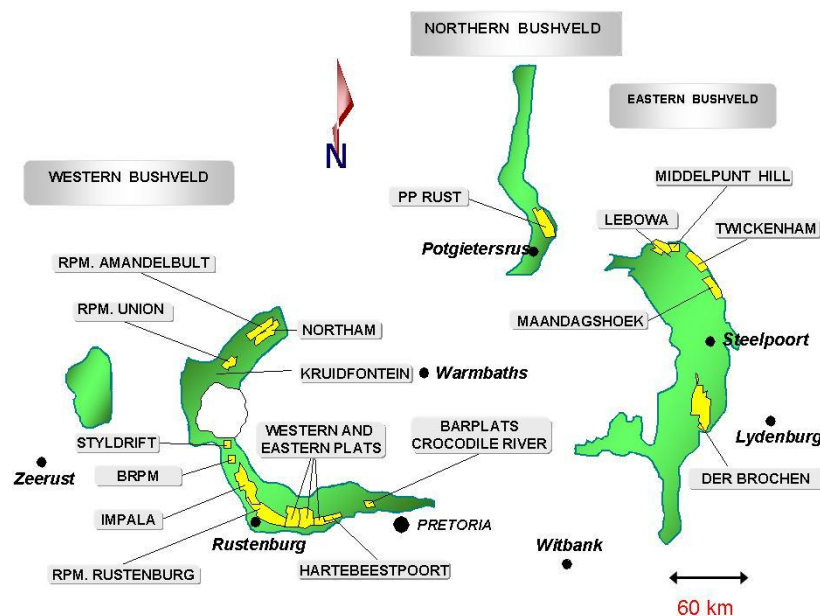


Figure 1-1 Map showing the western, northern and eastern lobes of the Bushveld Complex and the locations of the platinum mines

The Bushveld Complex is divided into the eastern, northern and western lobes as shown in Figure 1-1. The figure also presents the locations of the larger mines in relation to major towns. Traditionally, most of the platinum mining has taken place in the western lobe. However, recently mining activity has increased in the eastern Bushveld.

1.2 Typical geology around the Merensky Reef

The research that this thesis reports on concentrated on the Merensky Reef.

Typically the geotechnical conditions above the Merensky Reef are good, with gradational contacts between the rock types. The first sharp contact (possible parting plane) is at the base of the Bastard Reef, which occurs between 12 m and 30 m above the stopes. A general stratigraphic column showing the typical sequence of rock types above and immediately below the Merensky Reef is provided in Figure 1-2. The thicknesses of the lithologies differ considerably across the Bushveld, with the base of the Bastard Reef varying between about 10 m and 30 m.

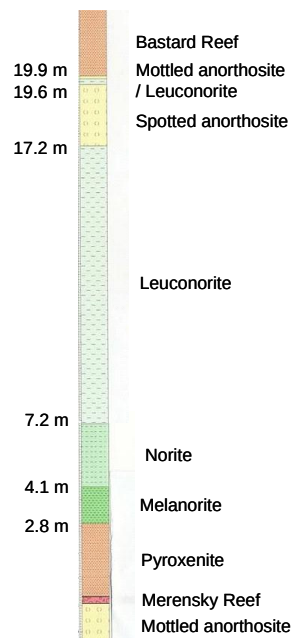


Figure 1-2 Stratigraphic column showing the typical rock sequences above the Merensky Reef

Dome structures are often observed in stope hangingwalls. These structures have been shown by Perritt and Roberts (2007) to be thrust structures that probably developed when the Bushveld strata deformed into a “basin” shape. The structures normally follow a weak plane such as the chromitite at the top of the Merensky Reef. However, occasionally the structure may dip up into the hangingwall, forming a dome- or a half-dome shape if the structure locates in a higher plane of weakness. These discontinuities complicate mining, and many of the falls of ground (FOG) accidents have been associated with these structures. However, the current study does not deal with this issue.

Potholes are a frequent phenomenon on the Merensky Reef. These geological structures are generally oval in plan (Figure 1-3) and comprise a sudden drop in the elevation of the reef within the confines of the oval. For this reason, a stope mined through such a feature and that does not follow the displaced reef would be mined in the unmineralised hangingwall strata. This off-reef mining often occurs in small potholes where the diameters are less than about 10 m. However, in regions of large potholes (>100 m), stoping is done at the lower reef elevation. In essence, the Merensky Reef cuts down steeply through the leucocratic (anorthosite and leuconorite) rocks in the footwall, usually until it reaches a more mafic horizon (pyroxenitic). This horizon may form the base of the pothole or a step in the side of a larger pothole that plunges deeper into the footwall and whose base may be the next mafic layer or even one further down the succession (Figure 1-4). Usually where the Merensky Reef comes to lie conformably on a pyroxenitic footwall stratum, the lower pyroxenite is pegmatised and mineralised to form a payable reef.

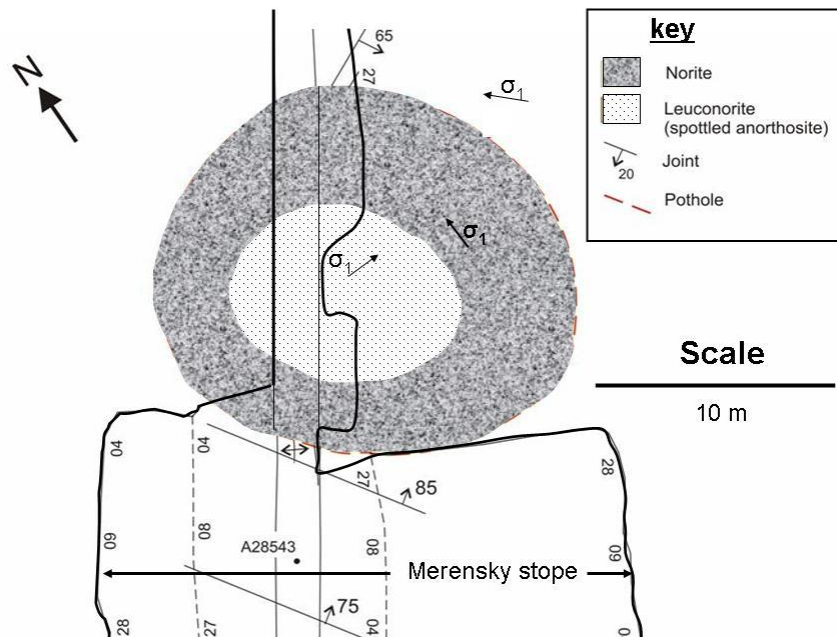


Figure 1-3 Plan view of a pothole on stope elevation

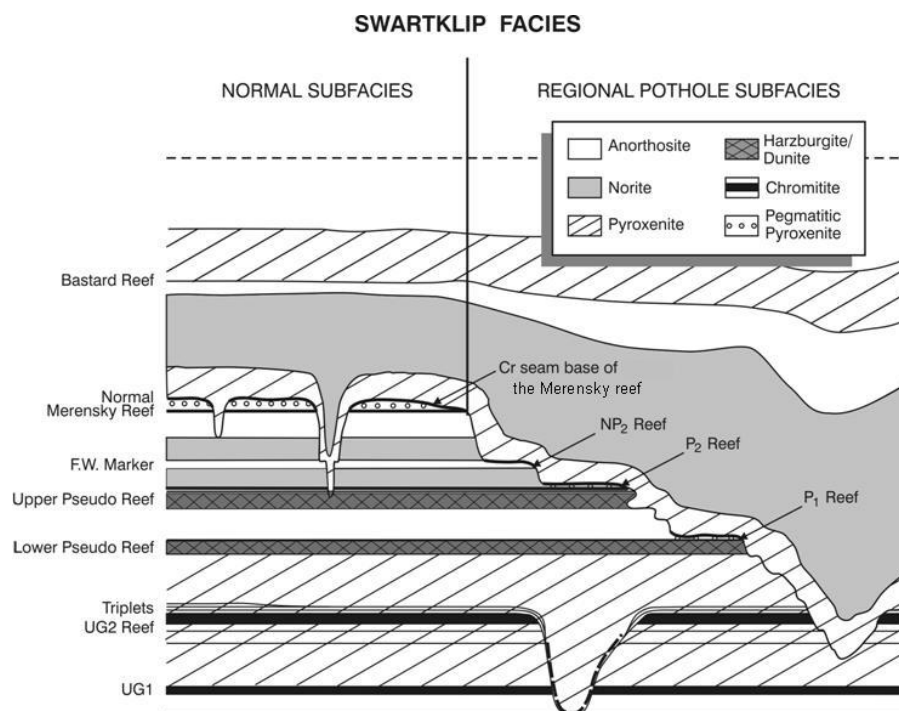


Figure 1-4 Potholing in the Swartklip facies of the Merensky Reef (modified after Viring and Cowell 1999)

Stress measurements conducted in a small pothole (<20 m, Figure 1-3) indicated exceptionally high horizontal stresses of around 100 MPa at 250 m below surface (Watson et al, 2005b). However, few other measurements have been done to determine the stress conditions in the various forms in which potholes occur. The stress states are likely to be different in other localities or in the larger potholes. The focus of the research done for this thesis concentrated on the rock mass behaviour around stopes and pillars and little research was done on potholes.

1.3 Stope layouts

The platinum group minerals in the Bushveld Complex are concentrated in two extensive shallow-dipping, tabular orebodies known as the Merensky and UG2 reefs. The dips of the reefs generally vary between 9° and 25° and, in mining the reefs, the strength of the host rocks generally allows relatively large, stable stope spans to be developed.

Historically, stope spans on the Bushveld Complex mines have been designed according to what worked in the past and are primarily based on economic and practical considerations (Swart et al, 2000). Stopes consist of several panels, each of which is typically about 30 m wide. These panels are separated by stable, 'crush' or 'yield' pillars, as shown in Figure 1-5. The most common mining configuration is breast mining, where the mining face is advanced on strike (Figure 1-5). However, up-dip and down-dip mining is also done for various reasons and the pillar lines are then oriented on dip. Regional pillars are designed to carry the overburden to surface.

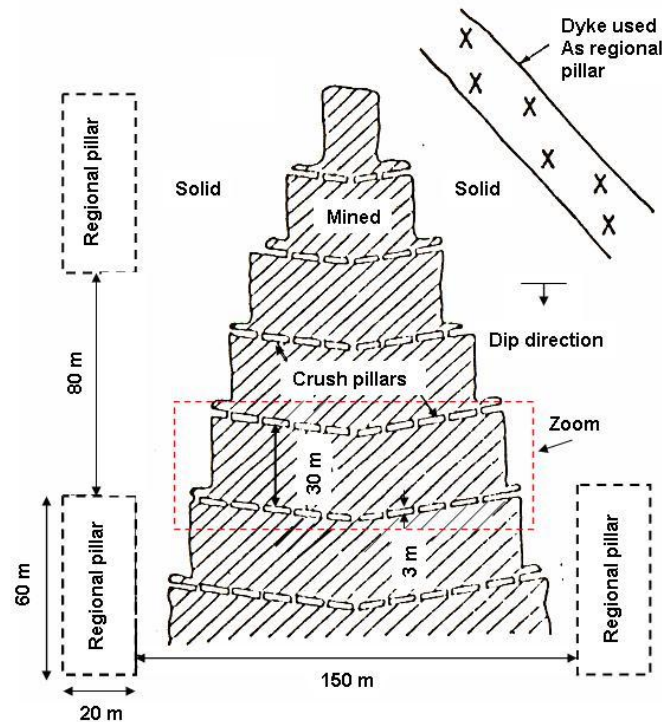


Figure 1-5 Plan view of a typical breast configured stope on the Merensky Reef (after Özbay and Roberts, 1988)

An enlargement of the zoom area in Figure 1-5 is provided in Figure 1-6, where the directions of photographs 'A' to 'F' that make up the photographs in Figure 1-7 are shown. Figure 1-6 and Figure 1-7 show typical scenes in a Merensky Reef breast stope. The advancing face is shown as 'A' in the figures. A side view of a crush pillar is provided in 'B'. The photograph was taken from a holing between the chain pillars. Pillar sizes vary but are commonly about 2.5 m to 3 m wide and 4 m to 6 m long in a 1.2 m-high stoping width. However, stoping widths also vary between 0.8 m and 1.6 m and pillar widths are adjusted to w/h ratios of about 2.5. A typical scene immediately on the down-dip side of a line of 'crush' pillars is shown in 'C', while 'D' shows a view of a panel, looking up-dip from the down-dip side. Gullies are cut on the down-dip side of panels for breast conditions and at the centre of a panel for up- and down-dip mining, respectively. These gullies are used for cleaning the face and as an access way for men and materials. A typical gully for shallow depth (<650 m below surface), breast mining scenarios is shown in 'E'. These gullies are generally mined slightly ahead of the breast face and are known as "advanced strike gullies" (ASGs). At greater depths below surface, it is common to extend the panel down-dip of the gully, as shown

in 'F', to keep the fractured stope hangingwall and pillar sidewall away from the access way. This extension is known as a "siding" and is usually cut between 0.5 m and 3 m on the down-dip side of the gully.

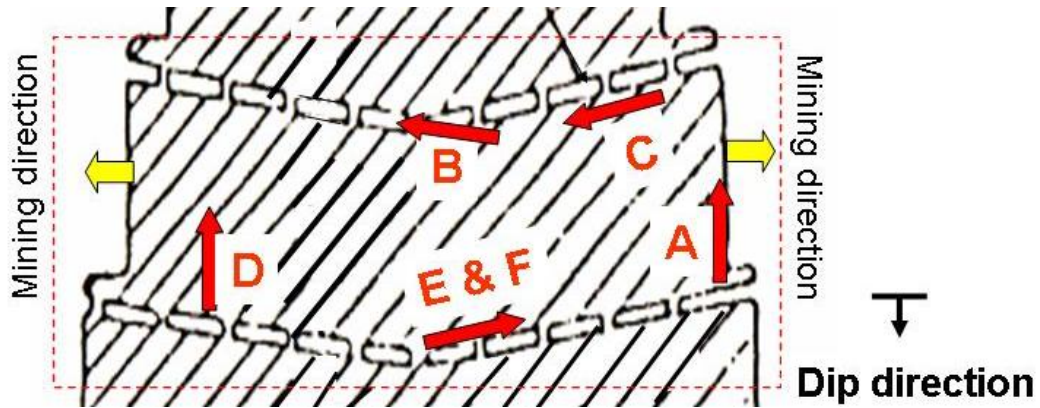


Figure 1-6 Zoom into Figure 1-5, showing the directions of the photographs in Figure 1-7

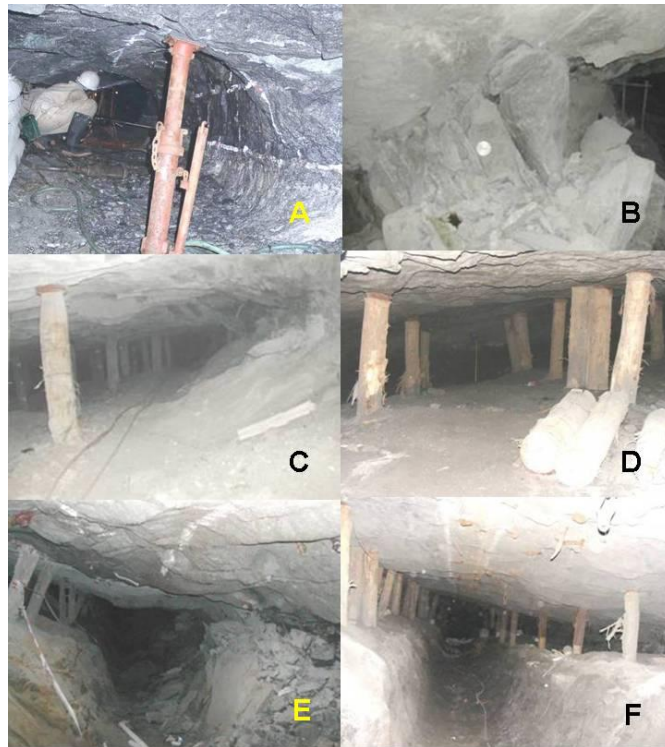


Figure 1-7 In-stope views of: A – advanced strike gully without sidings, B – ‘crush’ pillar, C – stope below pillars, D – into stope from the advanced strike gully, E – breast face and F – advanced strike gully (ADG) with sidings

Research into maximum panel spans between 'yield pillars' was conducted at Impala prior to the year 2000 (Kotze, 2002). The results indicated that relatively large spans (>40 m) were normally feasible on the Merensky Reef. However, 30 m-wide panels are considered favourable from an economic and practical mining point of view and this span is, therefore, most commonly used. The Kotze (2002) investigations were conducted where the standard support systems employed on the mines were used. These comprised 180 mm-diameter timber profile props, spaced 2 m apart on dip and strike.

1.4 Pillars used on the Bushveld platinum mines

Özbay et al (1995) reviewed hard rock pillar designs in use in South Africa in the mid 1990s. They exposed a considerable range in design practices (including the choice of pillar type) and uncertainty about key issues, notably the estimation of *in situ* strengths. Four common types of pillars are used on the Bushveld platinum mines at shallow to intermediate depth:

- Barrier/stability pillars, intended to remain elastic during the life of the mine (minimum width/height (w/h)) ratios usually about 10) for regional stability.
- Intact in-stope pillars, intended to remain elastic (unfailed) during the life of the mine, generally with w/h ratios greater than four. These pillars are required to support the overburden to surface and provide local stope stability. They are used mainly at shallow depths.
- Yield pillars, calculated to have safety factors equal to or greater than unity when cut (w/h ratios ranging between 3 and 5). Back-analysis and underground measurements have shown that these pillars often appear to punch into the footwall rather than yield elastically (Lougher, 1994).
- 'Crush' pillars, designed to 'crush' under stable, stiff loading environments near the face (w/h ratios typically ranging between 1.7 and 2.5). Their

main purpose is to prevent backbreaks (Jager and Ryder, 1999). If designed and cut correctly they have little potential for dynamic failure.

The curves shown in Figure 1-8 illustrate the behaviour of each type of pillar.

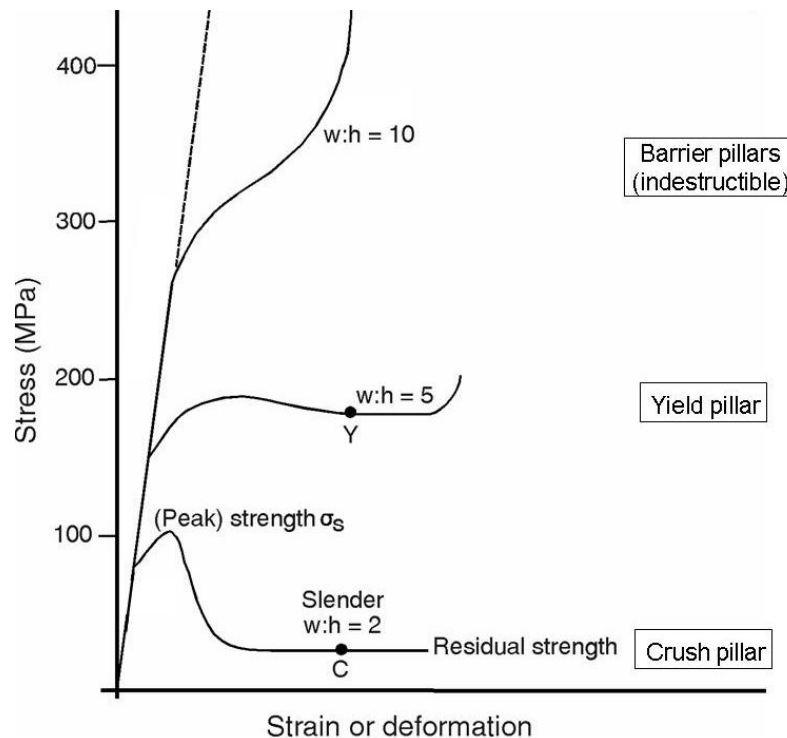


Figure 1-8 Illustrative stress-strain behaviour of various pillar types, showing the change in behaviour for different ranges of pillar w/h (after Jager and Ryder, 1999)

The ‘crush’ and ‘yield’ pillars are designed to maintain rock mass stability above the Merensky Reef to the base of the overlying Bastard Reef, which forms a parting plane up to 30 m above the reef. The pillars also reduce the demand on local stope support by restricting the height of the vertical tensile zone. Where ‘crush’ or “yield” pillars are used, regional stability pillars are appropriately spaced to carry the overburden to surface. These stability pillars are often located at the stope boundaries. In many cases, regional stability pillars are opportunistically sited around potholes or dykes.

Jager and Ryder (1999) suggest that there is a transition zone of approximately 300 m to 600 m below surface within which ‘crush’ pillars cannot readily be used. At these depths below surface the pillars may remain intact when cut and only

crush when loaded in the back areas, triggering violent failure or a potentially hazardous pillar run. At shallower depths, pillars are required to stabilise the surface and should therefore support the overburden to surface. In deeper stopes of about 600 m to 1000 m below surface the continued use of intact pillars is possible but only at reduced extraction ratios because of the heavier cover loads borne.

The behaviour of foundations is an integral part of pillar behaviour and must be included in pillar design (Watson et al, 2008). The pillar, hangingwall and footwall must be considered as components of an integrated and interacting system. To design these systems effectively, rock and rock mass behaviour must be understood. Lougher (1994) and York et al (1998) also show that pillars and stopes comprise an interactive system. Ryder et al (2005) demonstrated that the height of the vertical tensile zone above a stope is dependent both on the stress on the pillars and abutments surrounding the stope and/or the span between these pillars. The behaviour of pillars is, therefore, an integral part of stope stability. The mutual dependence of the stope and the pillars means that the two had to be studied together in the current research.

The design of stable stopes and pillars is one of the critical issues in mine planning. However, the process is currently not optimised on the Bushveld platinum mines and largely depends on empirical methods that make use of back-analysis techniques. As a result of this lack of optimisation, over- or under designing of excavations and pillars does occur, which has implications for extraction ratios and ore reserves. In addition, oversized excavations and incorrectly designed pillars have serious implications for safety.

1.5 Aims of the research

A series of pillar bursts that occurred in 2004 on the Merensky Reef with serious consequences has refocused attention on pillar design. Research results from the gold mines are not necessarily transferable to the platinum mines because of the different, often more ductile, nature of the Bushveld rocks. The discontinuities are also different, being much weaker and often serpentinised on the Platinum

mines. In addition, the virgin horizontal stress conditions on the platinum mines are often much higher than on the gold mines, even at intermediate depth.

The current investigation aimed primarily to study the behaviour of ‘crush’ pillars on the Merensky Reef. However, the literature and modelling, conducted as part of this study, showed that the rock mass around a stope influences pillar behaviour. Pillars also influence the behaviour of the rock mass around the panels, particularly where foundation failure occurs. The mutual dependence of the stope and the pillars meant that the two had to be studied together in the research that this thesis reports on. The rock mass surrounding Merensky Reef stopes was examined with the use of underground measurements of stress and deformation; and the interaction between pillars with their foundations (hangingwall and footwall) were studied in detail.

The intention of the research was to provide the South African platinum industry with guidelines on ‘crush’ pillar design for the Merensky Reef. The provision of these guidelines entailed an understanding of peak and residual pillar strengths, primarily from underground measurements. In addition, laboratory tests, standard analytical solutions and numerical models were used in the final analysis.

1.6 Summary of the research

This thesis describes the behaviour of the rock mass and pillars in three Merensky stopes monitored at RPM Amandelbult Section, Impala Platinum Mine and RPM Union Section respectively (Figure 1-1). The rock mass conditions at each site are described using the Q-system (Barton, 1988) and a modified version of the Stability Graph System (Watson, 2003) in Chapter 2.

The “New Stability Graph System” (Watson, 2003) was established from a database of stable and unstable hangingwall conditions on the Bushveld platinum mines. Definite clustering of the stable and unstable conditions is observed when the data is plotted against a measure of excavation size (Figure 1-9). Most of the data in Figure 1-9 were collected from Merensky panels and, therefore, the

system is well suited to the evaluation of panels on the Merensky Reef and the instrumentation sites in particular.

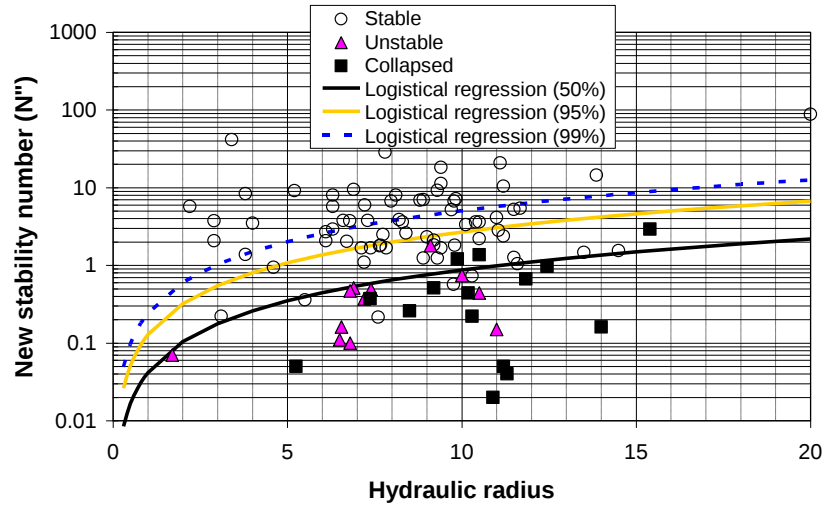


Figure 1-9 New stability graph showing the clustering of stable and unstable/collapsed panels

The underground sites used in the study are described in Chapter 2. The strategy adopted for this research was to measure the rock mass and pillar behaviour underground and to use numerical models and analytical solutions to interpret the measurements. Stress change measurements were conducted well above pillars and the starting stresses were established with field measurements in the same boreholes. Average pillar stresses (APS) were calculated from these measurements using Boussinesq equations (Poulos and Davis, 1974) and numerical models. In this way peak and residual pillar strengths as well as pillar behaviour were established.

Nonlinear rock behaviour was first observed in rocks from Impala Platinum Mine and was investigated with a microscope and a series of laboratory tests. Subsequently, this behaviour was also recognised at the other two sites, but only under certain drilling conditions. The results of the investigation, as well as a method of interpreting stress measurements in this material are discussed in Chapter 3. The influence of this rock behaviour on stopes is shown in Chapter 4.

Investigations of the rock mass behaviour around the stopes are discussed in Chapter 4. In the Chapter 4 studies, stress change measurements were conducted in the hangingwall of a stope to determine the hangingwall behaviour ahead of and behind an advancing stope face. In addition, stress change was measured over 'crush' pillars primarily to determine pillar stress change but also to observe the interactions of the pillars with their foundations. Several series of field stress measurements were conducted in vertical boreholes drilled vertically up into the hangingwalls, at the centre of the three instrumented panels. These measurements were taken at small intervals, to establish the horizontal stress profiles. All the stress measurements were interpreted together with extensometer, closure and closure-ride measurements, borehole camera surveys, and laboratory tests to determine the rock mass behaviour around the stopes.

The effects of the nonlinear rock mass discussed in Chapter 3 appear to have had little direct influence on pillar behaviour as the research showed that the rock mass became nonlinear only at stresses below about 10 MPa. However, the rock mass surrounding the pillars was affected where the virgin stress levels were high enough to cause micro-fracturing between different minerals during stress relaxation.

Peak pillar strength equations were developed from a Maximum Likelihood statistical back-analysis of stable and failed pillars from Impala Platinum Mine. These equations were primarily formulated to validate measured peak pillar strengths at Impala Platinum Mine, but also provided a good correlation with the measurements at the Union site.

Pillar stress-strain behaviour was measured during pre- and post-failure at all three instrumentation sites, providing peak and residual pillar strength results. The residual strength measurements were confirmed by a series of field stress measurements conducted across each of these pillars to determine the stress profiles in their residual states. The investigation results are discussed in Chapter 5. A profile showing the relationship between w/h ratio and residual pillar strength was established from the underground measurements, laboratory tests, analytical solutions and numerical modelling.

The findings of the research were used to formulate a design procedure for 'crush' pillars. This procedure is also discussed in Chapter 5.

2 Site description

This chapter describes the mining sequence and geotechnical conditions of the three instrumentation sites. The chapter includes issues that influenced rock mass behaviour such as: pillar shapes, stoping width, extent of external mining and geomechanical properties of the rock. A breast mining configuration was employed at all three sites.

2.1 Amandelbult 1-shaft

The instrumentation site (Amandelbult site) was located at a depth of 600 m below surface and is shown in the stope sheet provided in Figure 2-1. The investigations were conducted in panels 12-16-5E and 13-16-1E. The red areas in the figure represent the mining that occurred before instrumentation and the subsequent face advances are shown by a series of colours. The grey shows the unmined areas and pillars.

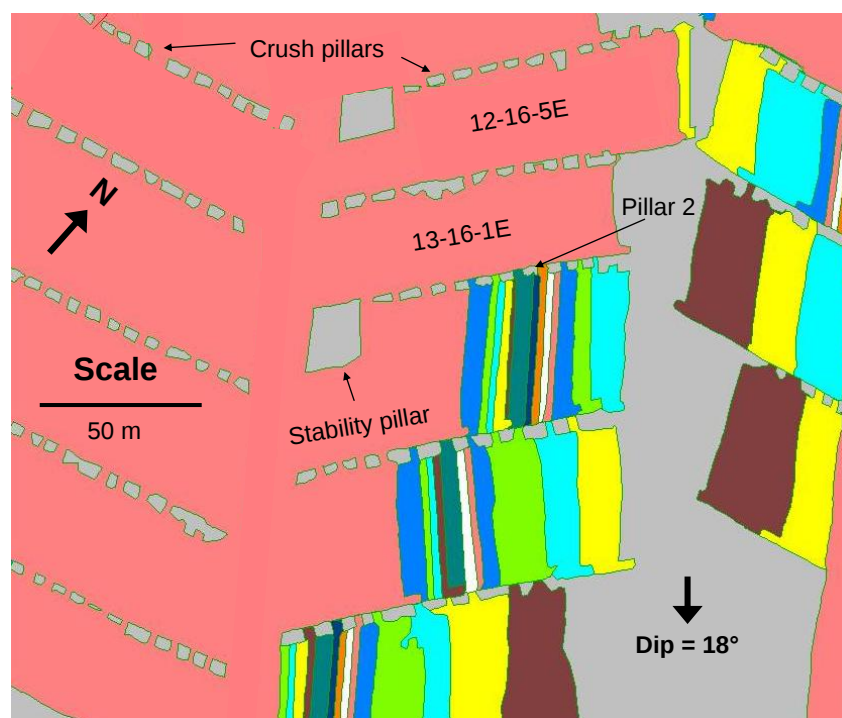


Figure 2-1 Plan showing the Amandelbult instrumentation site

The standard 'crush' pillar size at the site was 3 m x 4 m in the dip and strike directions, respectively, with a stopping width of about 1.1 m. Figure 2-1 shows the disciplined pillar cutting that was observed. No sidings were left between the pillars and the ASG. Instead, the up-dip sides of the pillars were inclined at between 70° and 80°, dipping in the opposite direction to the strata and thus preventing fractured slabs from rotating into the ASG (Figure 2-2 and Figure 2-3). The immediate footwall of the Merensky Reef at the site is a brittle anorthosite, which has a significantly higher uniaxial compressive strength (180 MPa) than the immediate hangingwall (100 MPa) (Figure 2-4). This anorthosite is exposed on the down-dip side of some of the pillars and in the ASGs on the up-dip sides of the pillars.

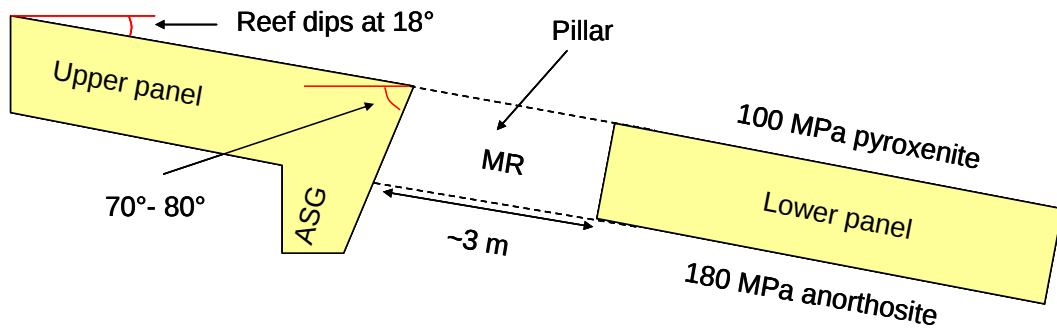


Figure 2-2 Amandelbult site: diagram showing a section through a 'crush' pillar (not to scale)

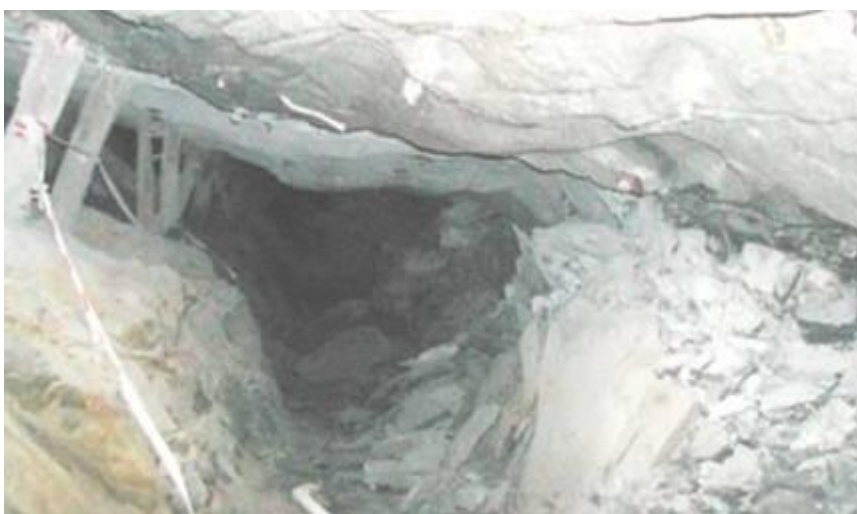


Figure 2-3 Amandelbult site: ASG on the up-dip side of the pillars

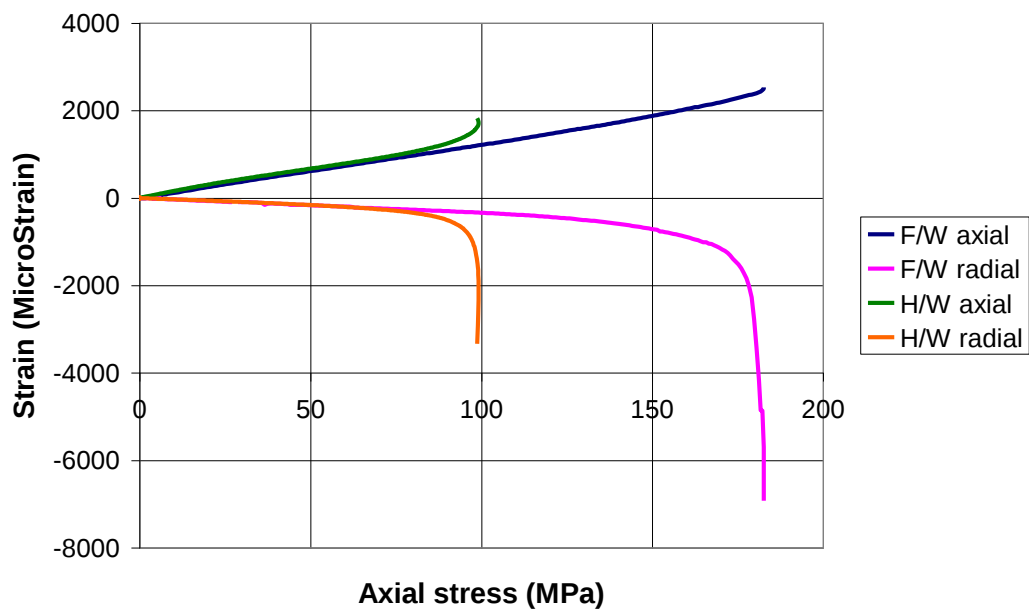


Figure 2-4 Amandelbult site: stress-strain curves for immediate footwall (anorthosite) and hangingwall (pyroxenite) rocks

The residual strength of the ‘crush’ pillars is required to support the rock mass up to the base of the Bastard Reef, some 20 m above the stope. Regularly spaced 15 m x 15 m regional stability pillars are designed to support the strata to surface. Two such stability pillars were cut in the instrumented stope as shown in Figure 2-1.

The immediate hangingwall could be described as good (Barton, 1988) with two orthogonal, vertical joint sets as shown in Figure 2-5. The strike lengths of most joints in both sets were less than 6 m. Thin coatings of serpentinite and calcite were observed on the surfaces of both sets, mostly less than 0.5 mm thick. All the observed discontinuities were tightly closed and dry. Rock mass evaluations were performed using the Q-system (Barton, 1988) and a modified version of the Stability Graph System (Watson, 2003). The results are shown in Table 2-1 and plotted on the New Modified Stability Graph in Figure 2-6. The investigations suggest stable conditions for this panel.

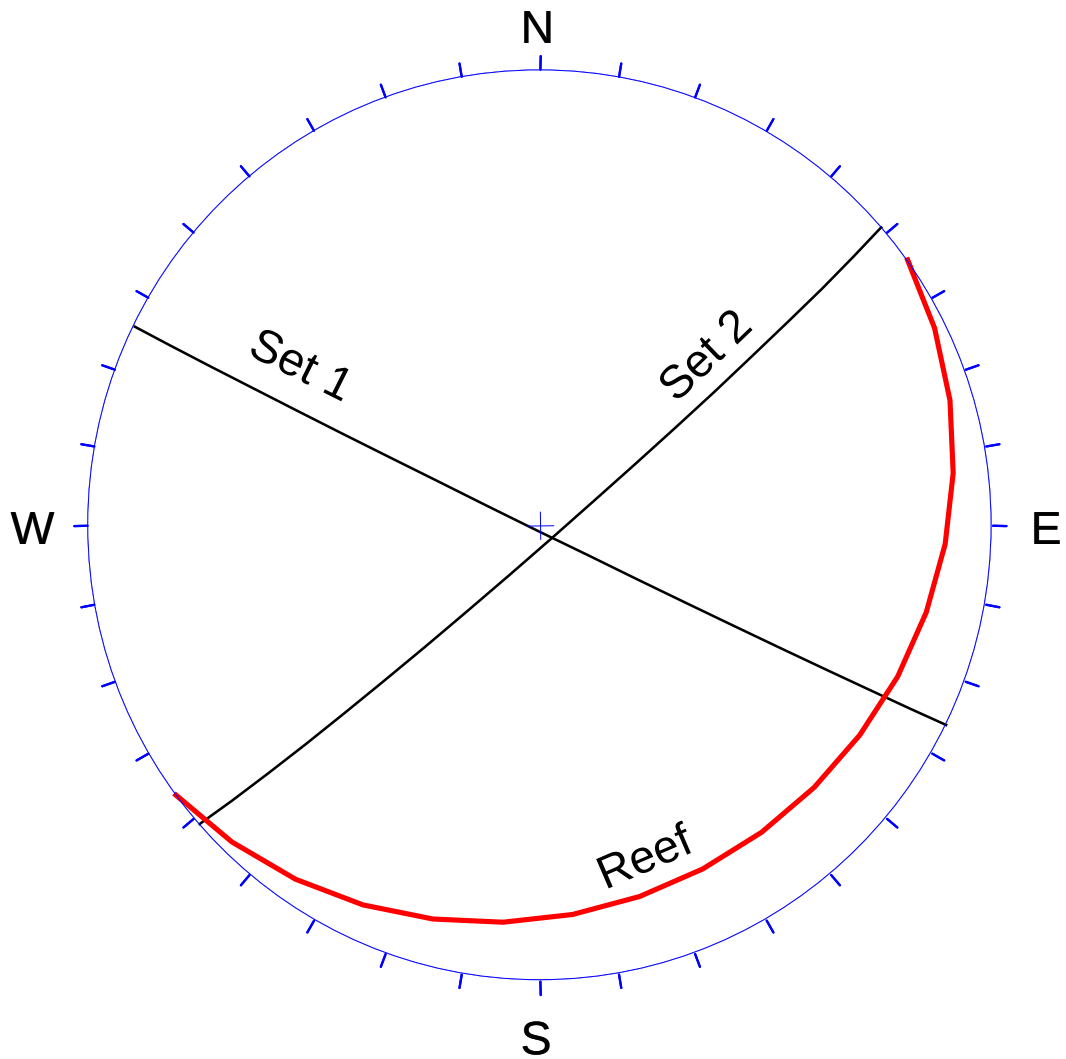


Figure 2-5 Amandelbult site: Schmidt, lower hemisphere, equal area stereo net showing the orientations of the joint sets above the 12-16-5E panel.

Table 2-1 Amandelbult site: results of the rock mass rating evaluations

Rating system	Rating	Description
RQD	100	Excellent (Barton, 1988)
Q	18.3	Good (Barton, 1988)
N"	39.5	Stable (Watson, 2003)

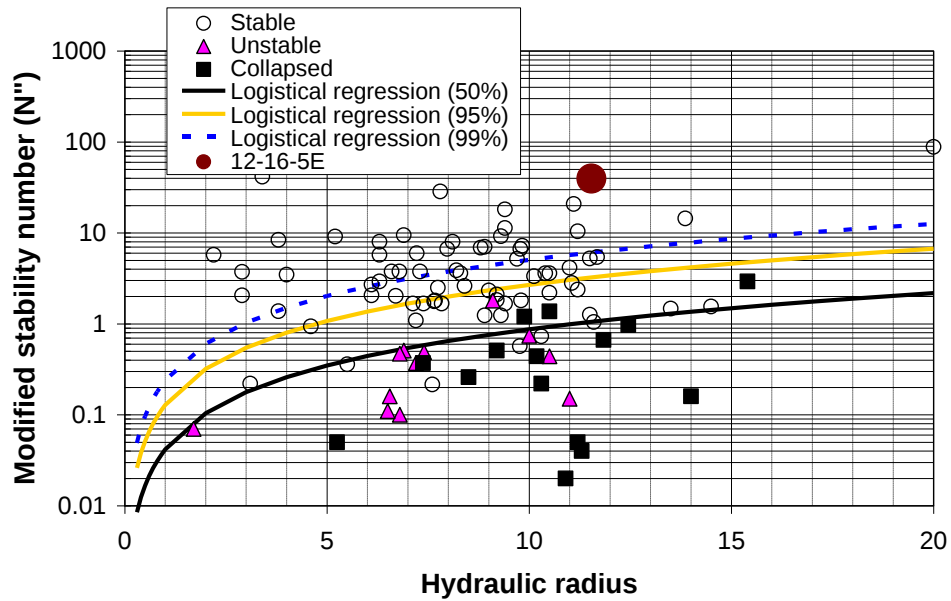


Figure 2-6 New modified Stability Graph showing the relative plot of Panel 12-16-5E at the Amandelbult site

The stratigraphy between the Merensky and Bastard reefs is shown in Figure 2-7. One rock type grades into another and no potential parting planes were apparent between the strata up to the base of the Bastard Reef some 20 m above the stope. At the base of the Bastard Reef there is a “weak” chromitite stringer along which parting could take place. However, the often observed calcite-filled discontinuities were not seen in the boreholes that were drilled through this stratum. A general k-ratio of unity was established in this area of the mine by stress measurements performed by the mine (Van Aswegen, 2008). Circular blasthole sockets observed in the development tunnels also suggested this ratio (Watson et al, 2005a).

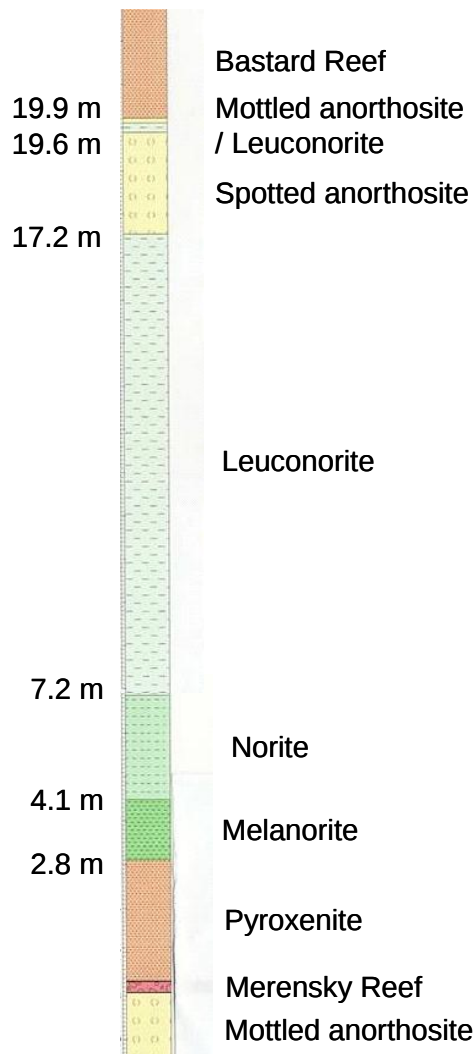


Figure 2-7 Amandelbult site: stratigraphic column showing the Merensky hangingwall stratigraphy above the 12-16-5E panel

The height of the parting plane at the base of the Bastard Reef is greater at Amandelbult than in the mines in the Rustenburg area (Figure 1-1). However, the rock mass conditions and mining layout are fairly typical of shallow depth, good rock mass conditions that do not have the dome structures described by Perritt and Roberts (2007).

2.2 Impala 10-shaft

The instrumentation site (Impala site) was located in Panel 1866/7s at a depth of 1100 m below surface. A stope sheet showing the instrumented panel and the surrounding stope is provided in Figure 2-8. The figure shows the mining sequence and monthly face advances during the site investigations. The area represented by red was mined prior to the study and the grey areas were unmined or pillars. Initially the 1866/7s panel was mined ahead of the surrounding panels, thus forming an isolated panel. This mining configuration allowed for the monitoring of the rock mass behaviour without the influence of the crush pillars, and provided opportunity to determine the degree of inelastic rock behaviour. The 7s face was stopped when it reached stage 3 (two advances back from the final position shown in Figure 2-8). At this stage working conditions were extremely hot and it was necessary to improve ventilation by advancing the down-dip and up-dip panels. These two panels were advanced together, almost in line, until the position that is represented in Figure 8 by the dark blue colour (stage 7). At this stage the panel above (8s) was off-reef at the bottom end. The 8s face was stopped towards the end of March 2006 while holings were cut in the strip-pillar between 7s and 8s and a siding was cut in the up-dip side of Pillars 2 and 3. During the time that the 8s face stood, mining continued in the 7s panel to the final position marked on the plan, and Panel 6s (below 7s) was mined to the position represented by the orange colour (stage 8). When mining was resumed in Panel 8s, after a break of nearly three months, Panel 7s operations were temporarily halted and Panels 6s and 8s were mined simultaneously. The 8s face was mined for just over two months, at which stage poor hangingwall conditions forced this panel to stop permanently. However, the 6s panel continued and operations in the 7s panel were resumed. Both the 6s and 7s panels were mined to the adjacent raise (not shown on the plan). The panel above 8s was mined to its final position and collapsed before the instrumentation project started.

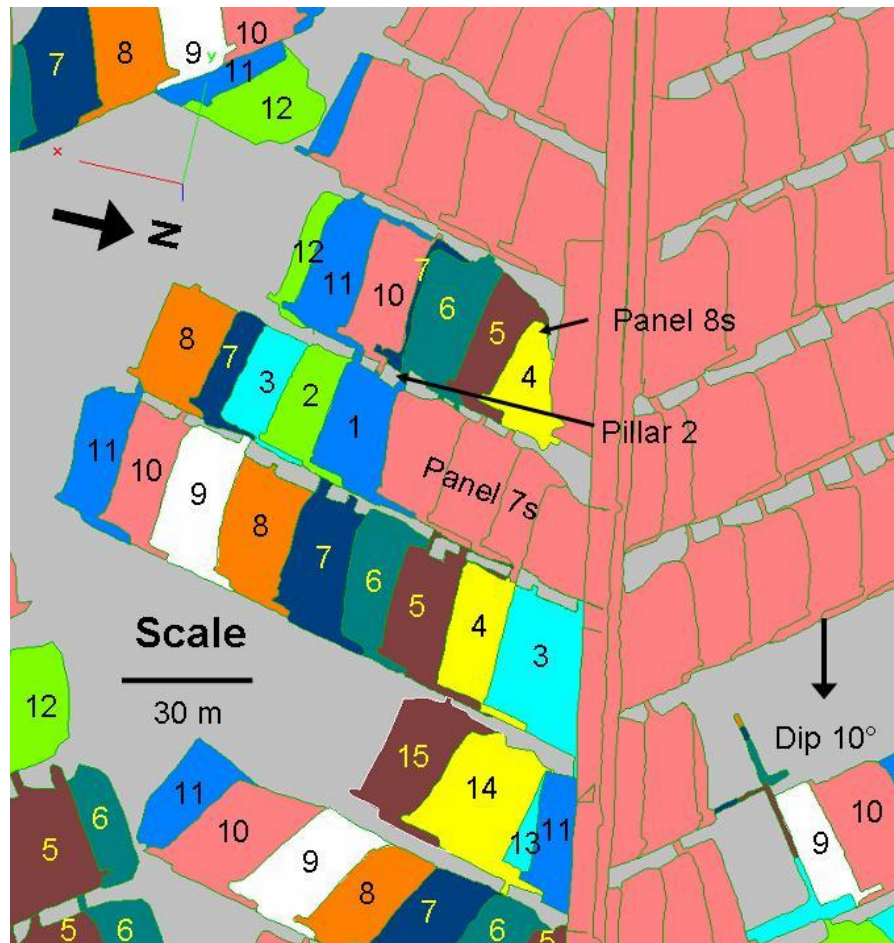


Figure 2-8 Plan showing the location of the Impala instrumentation site. The series of colours indicate the mining sequence and the grey background represents unmined areas and pillars

Only one, almost vertical, joint set was observed in the immediate hangingwall. The orientation was almost perpendicular to the reef dip as shown in Figure 2-9. A geotechnical survey was performed along a 5 m scan line conducted on strike about 5 m below Pillar 2 (Figure 2-8) and the results are shown in Table 2-2 and Figure 2-10. The rock mass could be described as very good (Barton, 1988). The many shallow and steeply dipping boreholes drilled across the hangingwall of Panel 7s for instrumentation installation confirmed the good conditions. No open discontinuities were observed in the hangingwalls except adjacent to the pillars.

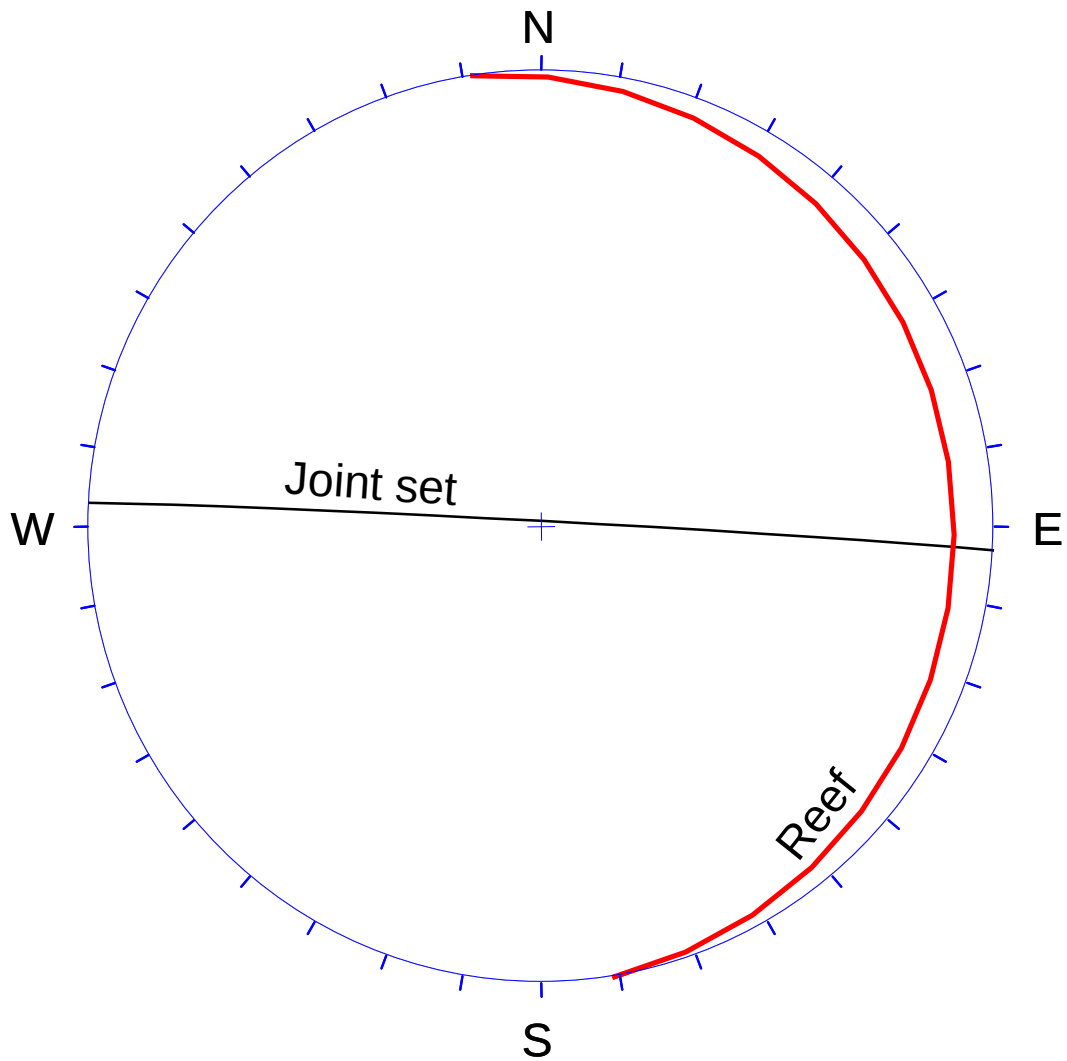


Figure 2-9 Impala site: Schmidt, lower-hemisphere, equal-area stereo net showing the orientation of the joint set and the reef

Table 2-2 Impala site: results of the geotechnical investigation above the instrumented panel

Rating system	Rating	Description
RQD	100	Excellent (Barton, 1988)
Q	50	Very Good (Barton, 1988)
N"	105	Stable (Watson, 2003)

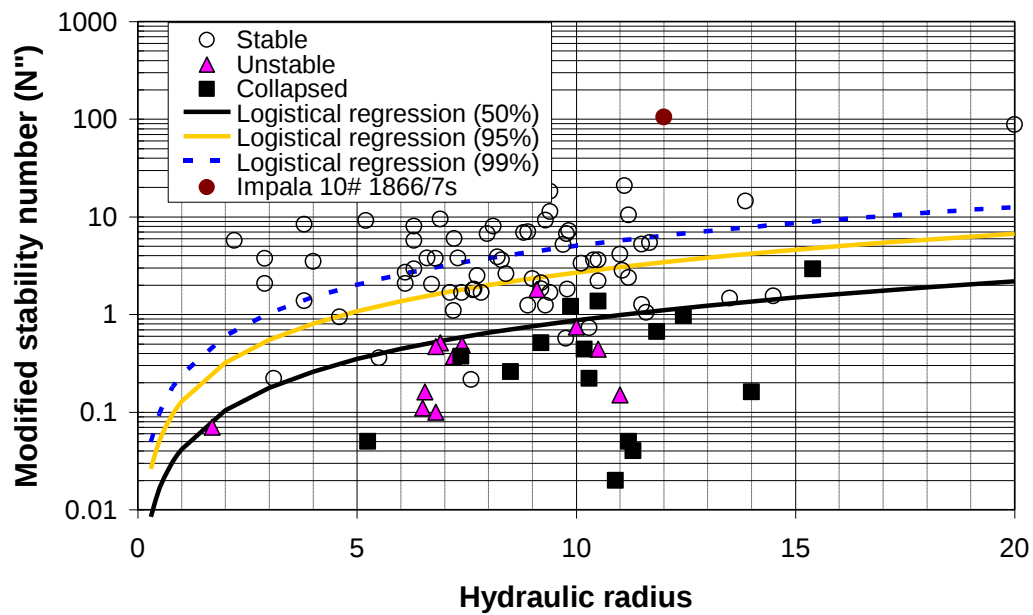


Figure 2-10 New modified stability graph plot of stopes supported on 200 mm diameter mine poles spaced 2 m x 2 m, showing the plot of the 1866/7s panel at the Impala site

A stratigraphic column showing the stratigraphy between the Merensky and Bastard reefs and the immediate footwall of the Merensky Reef is provided in Figure 2-11. The mineralisation is mostly in a 1 cm-wide chromitite stringer with a rapid drop in grade into the hangingwall and footwall materials. Thus the stope was cut about 0.6 m into the hangingwall and footwall rocks with the chromitite stringer, therefore, being in the centre of the pillars. The immediate hangingwall of the 7s panel was located in the Merensky pyroxenite lithology, with a thickness of about 0.5 m. The norite above this pyroxenite is known locally as the transition zone and was also about 0.5 m thick. The rock type known on the mine as spotted anorthosite (Figure 2-11) is actually a leuconorite and extends up to about 8.4 m above the stope. The MID3 mottled anorthosite extends to about 12.2 m. All the rock types in the immediate stope-hangingwall grade into one another and there are no definite parting planes up to the contact between the mottled anorthosite and the Bastard Reef at 12.2 m. A distinct change in rock type occurs at this height and a potentially weak plane is defined by a 1 cm-thick chromitite stringer at the base of the Bastard Reef. Parting could occur on this plane if panel spans are too large or if the residual strengths of in-panel pillars are too low.

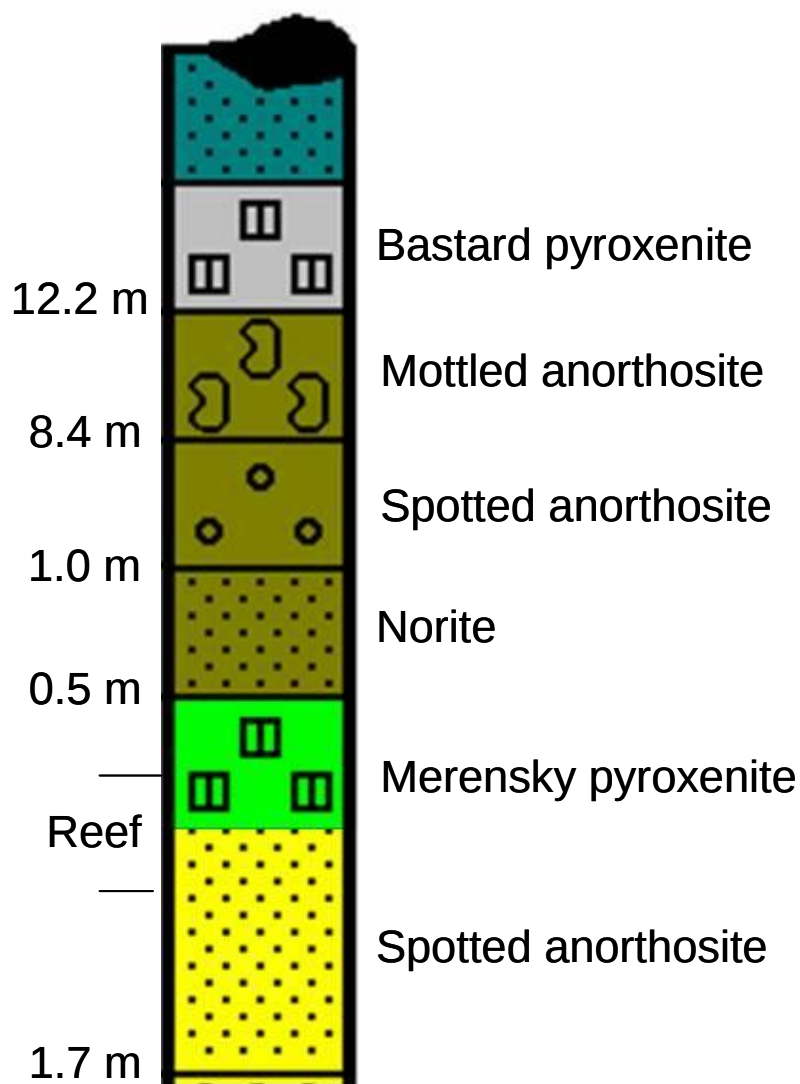


Figure 2-11 Impala site: stratigraphic column showing the rock types around the Merensky Reef. The mineralisation extends into the Merensky pyroxenite and spotted anorthosite rock types

An extensive stress measurement programme conducted across the mine suggests that the k-ratio in this area is about 1.3. The average vertical and horizontal dimensions of the sockets in the drive below the stope confirmed this ratio (Watson et al, 2005a). A comparison between direct stress measurements and numerical modelling back-analysis also suggests a k-ratio of about 1.3 in the leuconorites and anorthosites at the site.

The stope conditions and mining layout at the site were typical of good rock mass conditions at intermediate depth mining.

2.3 Union Section, Spud-shaft

Instrumentation was installed at the Union Section, Spud-shaft (Union site) to monitor the stope behaviour under “blocky” hangingwall conditions, with numerous shallow-dipping discontinuities (Figure 2-12). The measurements were split between two panels approximately 0.5 km apart on strike, both at a depth of about 1400 m below surface. At Site 1, the stress condition and deformation of an isolated pillar and associated deformations in the adjacent panel were monitored during and after excavation (Figure 2-13). The stope in Figure 2-13 appears to be a graben-like structure, bounded by a reverse fault and a dyke on the south and north sides respectively. Site 2 was used to determine the stress conditions in the hangingwall after stoping was completed (Figure 2-14). Face advances are denoted by different colours and the mining prior to the instrumentation installation is shown in red. The grey in both figures represents unmined areas and pillars.



Figure 2-12 Union site: blocky hangingwall adjacent to the isolated pillar

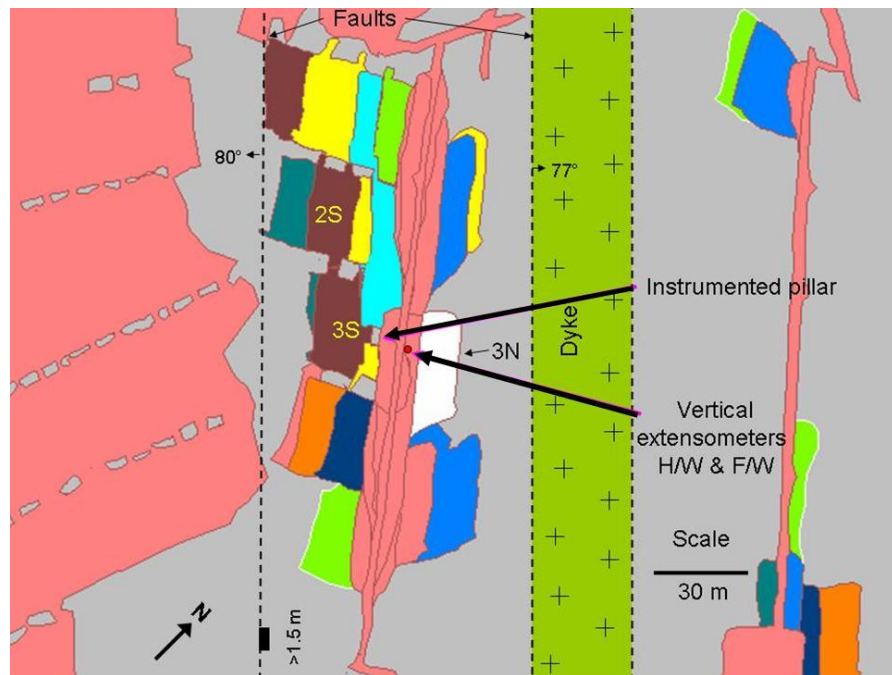


Figure 2-13 Union Site 1: stope sheet showing the location of the instrumented pillar. Mining steps are represented by the different colours. Stope on left mined prior to instrumentation installation



Figure 2-14 Union Site 2: stope sheet showing the location of the vertical hangingwall borehole used to measure the horizontal stress in the hangingwall of a completed stope

The sockets in the development tunnels suggest a k-ratio of about 0.5 in the pyroxenites (Watson et al, 2005a). The site was located in pothole reef and the footwall rock (Tarentaal) was heavily jointed. There was also a weak 1 m-thick Pseudo Merensky pyroxenitic lithology about 2 m below the stope.

A geotechnical evaluation of the hangingwall was conducted along a 5 m scan line oriented on dip, about 8 m north-east of the instrumented pillar. Four joint sets were identified together with random joints, some of which were shallow-dipping. The joint and reef directions have been plotted on an equal-angle, lower-hemisphere stereo net in Figure 2-15. The analysis shows the probability of wedge formation and, together with the shallow-dipping random joints, there is a strong possibility of unstable blocks being formed. The joint set oriented approximately parallel to the faults (Set 2) was closely spaced at about 300 mm apart. The other joints were spaced about 1.7 m apart, which together created a blocky hangingwall that could be described as poor rock mass conditions (Barton, 1988). The results of the evaluation are shown in Table 2-3 and Figure 2-16.

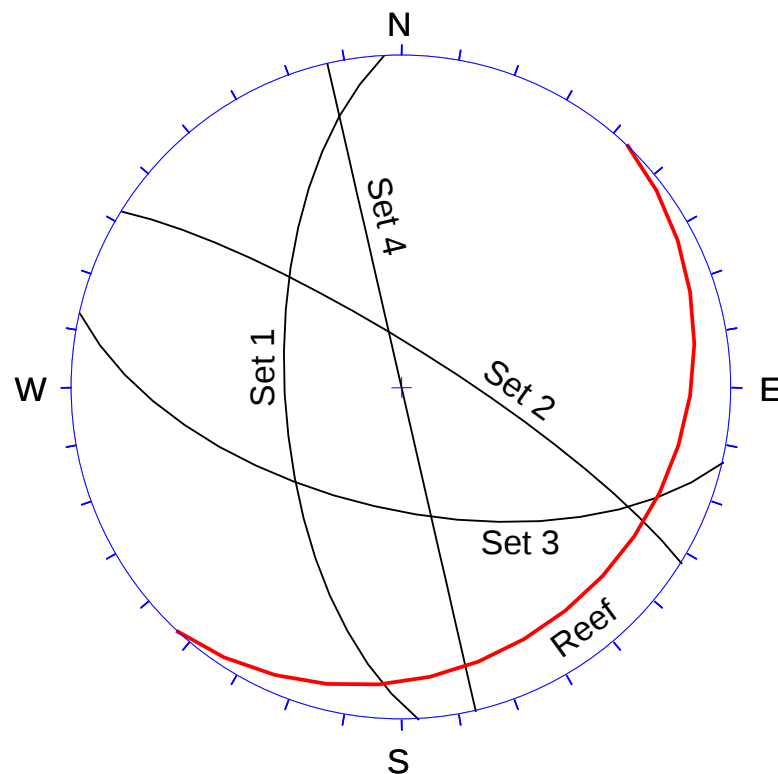


Figure 2-15 Union site: Schmidt, lower-hemisphere, equal-area stereo net showing the orientation of the joint sets and the reef

Table 2-3 Union site: results of the hangingwall geotechnical investigation

Rating system	Rating	Description
RQD	95	Excellent (Barton, 1988)
Q	3.2	Poor (Barton, 1988)
N"	5.6	Stable (Watson, 2003)

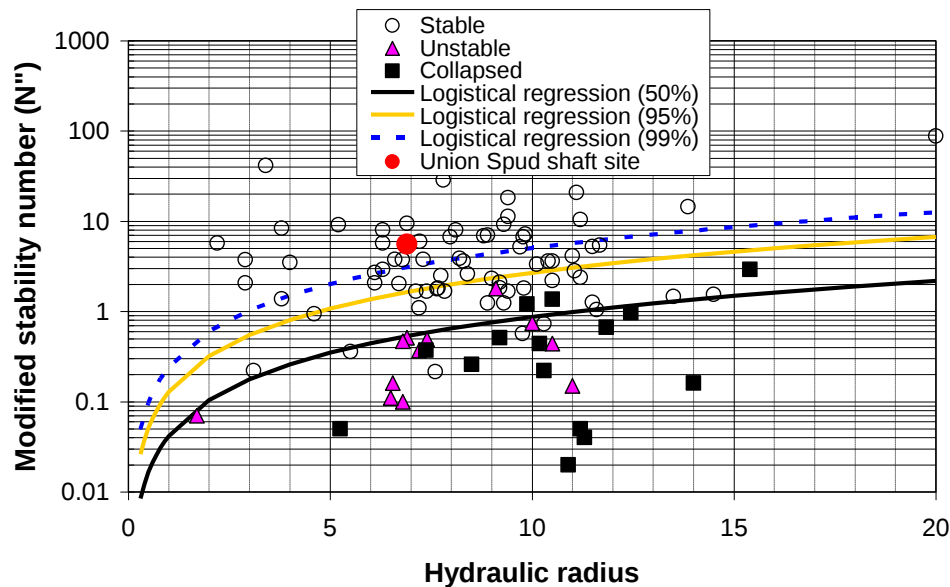


Figure 2-16 New modified Stability Graph plot of stopes supported on 200 mm-diameter mine poles spaced 2 m x 2 m, showing the plot of the 3n panel at Union Site 1

The stratigraphy between the Merensky and Bastard reefs is shown in Figure 2-17. One rock type grades into another and no potential parting planes were apparent between the stratigraphy up to the base of the Bastard Reef about 20 m above the stope. However, an abrupt change in rock type was observed between the norite and leuconorite at the site used to measure hangingwall stresses, which may have assisted in beam formation at that site. At the base of the Bastard Reef there is a “weak” chromitite stringer from which parting could take place.

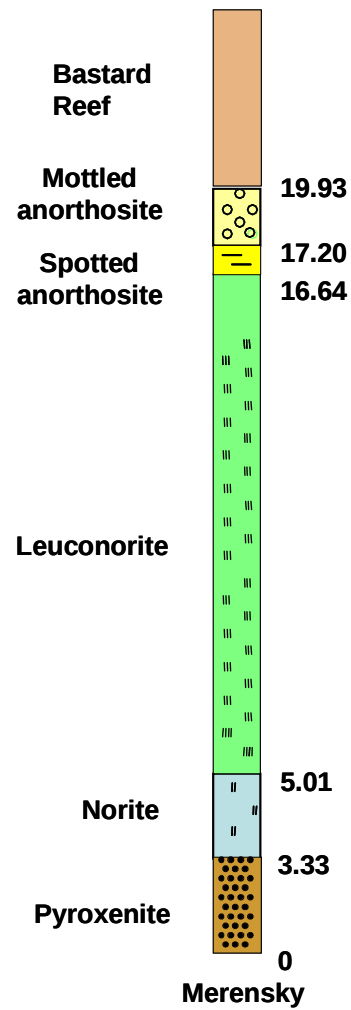


Figure 2-17 Union Site 1: stratigraphic column showing the Merensky hangingwall stratigraphy

The site provides information on rock mass behaviour in poor rock mass conditions at intermediate depth. However, the geomechanical conditions at the location were unusual in that the stope was located in a large pothole and the footwall rock was weaker than normal.

2.4 Summary

This chapter has provided a geotechnical description of the three instrumentation sites where most of the data discussed in this thesis was collected. The sites, described as follows, were classified using rock mass rating systems:

- Amandelbult – Shallow depth good rock mass conditions
- Impala – Intermediate depth good rock mass conditions
- Union – Intermediate-depth poor rock mass conditions

A historical account of the mining sequence during the time of the instrumentation installation and monitoring is also discussed. This chapter sets the scene for the research discussed in the following chapters.

Cylindrical-shaped rock samples were collected and geomechanically tested from boreholes drilled at all three sites. Some of the test results showed nonlinear stress-strain behaviour, which made the interpretation of stress measurements difficult. The nonlinear behaviour was noticed at all three sites, but only the Impala site was exclusively nonlinear. The following chapter discusses the nonlinear behaviour and describes a theory for its origins. The chapter also discusses why the nonlinear behaviour developed only in some samples at the other two sites. A methodology for interpreting strain-gauge-based stress measurements in this material is also included.

3 Rock specimen behaviour

The previous chapters described the three sites where the investigations for the current research took place and the setting of the sites in the context of the Bushveld Complex. The sites were classified as “shallow-depth good rock mass conditions” (Amandelbult site), “intermediate-depth good rock mass conditions” (Impala site) and “intermediate-depth poor rock mass conditions” (Union site), according to the depth of the workings below surface and geotechnical rock mass rating systems. This chapter deals mainly with the nonlinear geomechanical behaviour that was observed on all the rock specimens from the Impala site and in some of the tests from the Amandelbult and Union sites. The results of the laboratory investigations are used to describe the rock mass behaviour and to determine an evaluation methodology for stress measurements. The findings described in the chapter are also used to explain the rock mass measurements referred to in subsequent chapters.

3.1 Introduction

Uniaxial compressive strength (UCS) tests were conducted on rock specimens from each of the three instrumentation sites to determine the elastic constants required for stress evaluation. The cells used in the measurement of stress require the material elastic constants in order that the measured strains can be converted to stress. A separate UCS test was conducted for almost every stress measurement. In total, about 200 UCS tests were conducted in pyroxenite, norite, leuconorite and anorthosite rock types. Approximately 100 of these came from the Impala site, from both vertical and horizontal boreholes drilled above the pillars and panels. A strong nonlinear stress-strain relationship was observed in all the tests from the Impala site, including cores retrieved from both the vertical and horizontal boreholes. The nonlinear behaviour was also shown by the rock types that were tested from the footwall. At the other two sites, however, it appears that only the cores drilled under high stress conditions showed nonlinear behaviour, whereas cores drilled under low stress conditions (destressed by mining) showed linear behaviour. An example of the linear and nonlinear

behaviour in the UCS tests from the Amandelbult and Impala sites is shown in Figure 3-1. Note that strain is plotted against stress in the figure.

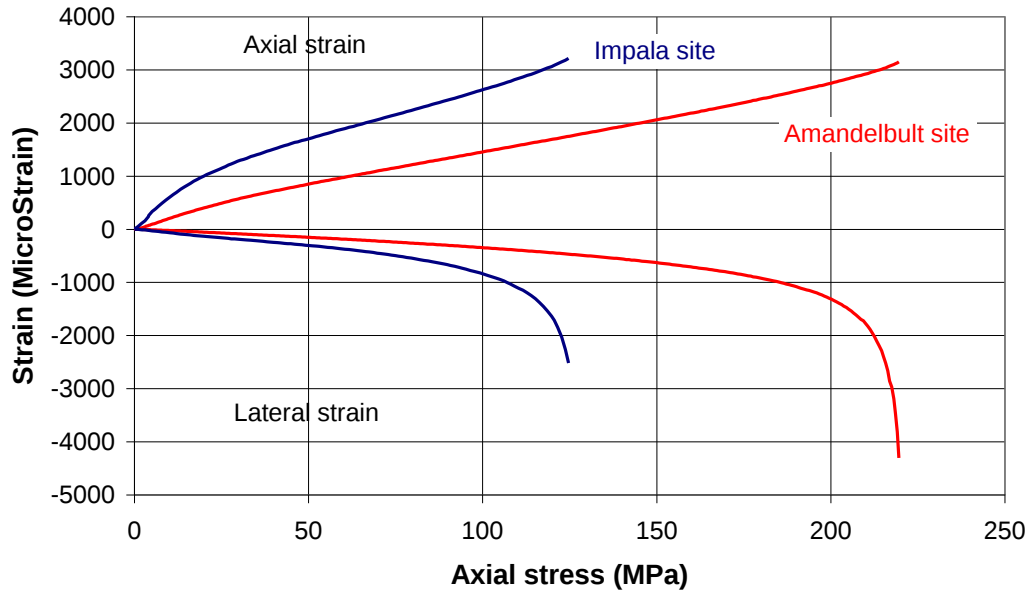


Figure 3-1 Typical uniaxial test results of linear and nonlinear anorthosites from the Amandelbult and Impala sites, respectively

The nonlinear samples are elastic but the tangential modulus changes at every point along a strain-stress curve (Figure 3-1). The main differences between the nonlinear and linear samples are described below:

- nonlinear samples have a significantly lower peak Tangential modulus (30 GPa) than the linear samples (83 GPa) (determined at the lowest slope of the axial strain-stress curve);
- the ratio of radial to axial strain in the nonlinear samples (conventionally considered as the Poisson's ratio) is significantly greater than in the linear samples, e.g. 0.7 and 0.32 at 50% of the UCS for the nonlinear and linear samples respectively; and
- the nonlinear samples are weaker (100 MPa) than the linear samples (180 MPa).

Thin sections from the Impala rocks were compared to linearly elastic samples of the same rock type from the Amandelbult and Union sites, and the results of the investigation are provided in Appendix A. The evaluation indicated that the

behaviour could not be associated with a difference in mineral composition. Also, no correlation was found between the density or orientation of micro fracturing and the nonlinearity of the stress-strain relationships. It was found through limited-scanning electron-microscope investigations, however, that there were open micro cracks in the specimens that exhibited nonlinear behaviour. These open micro cracks were not detected in specimens that exhibited a linear stress-strain relationship. Further investigations were conducted by interrogating a small database of UCS tests from Impala Platinum. It was found that cores that were retrieved at depths of less than 1000 m below surface did not demonstrate this nonlinear behaviour. A literature review was conducted to determine whether this condition has been observed elsewhere in the world and the possible causes of the condition.

Numerical models were run to simulate the effects of open cracks on sample behaviour. The investigation confirmed that the observed nonlinear behaviour is probably a product of open micro cracks. Subsequently, specialised tests were conducted in the laboratory to characterise the nonlinear behaviour.

Stress measurements were particularly difficult to interpret in the nonlinear rocks because the tangential modulus was different at every point along the stress-strain curve. It was therefore necessary to establish a methodology to determine stress from the measured strains under these conditions. In addition, it was important to know whether the observed behaviour in the laboratory tests were the same as in the rock mass; i.e. whether the rock mass was also nonlinear. Once a theory for the behaviour had been established, a solution describing the nonlinear component of the behaviour was determined. Nonlinear rock behaviour has implications for non-destructive measurements using seismic waves and determining seismic event sources. In addition, there are implications for mining, such as increased closure.

3.2 Petrographic investigations

Microscope and XRF studies were conducted to determine the cause of the nonlinear behaviour observed on many samples in the test laboratory. A detailed

description of the investigation is provided in Appendix A and the salient findings are discussed in this chapter. In particular, the following issues were investigated to determine any possible associations with the nonlinear samples:

- unusual minerals;
- crystal or micro-fracture alignment; and
- density of micro fractures.

None of the above could be definitively identified during the normal microscope and XRF investigations. However, it was established on a limited number of samples, at high magnifications and with the use of electron microscope (SEM) technology, that the nonlinear behaviour might have been associated with open micro fractures. A literature review was carried out to establish if such a relationship between open micro fractures and nonlinear behaviour has been observed elsewhere in the world.

3.3 Literature review: nonlinear elastic behaviour

The review revealed that nonlinear elastic behaviour in brittle rock is generally associated with the presence of open micro cracks. Gradual closure of these cracks, in response to loading in compression, results in compaction and an associated increase in effective stiffness. Walsh (1965) analyses the effect of individual cracks on the effective material stiffness, while Bristow (1960) and Kachanov (1992) use the concept of a crack density parameter. Unique experiments were conducted by Carvalho et al (1997) on artificially cracked aluminium plates to determine the effects of these cracks on the material behaviour. Carvalho et al's (1997) results showed good agreement with theoretical models, and these authors subsequently used the theory of non-interacting cracks to characterise micro cracks in a Charcoal Granite. With the use of a simple FLAC model, it could be established that the two-dimensional theoretical model provided a good match with a numerical model of a closing crack.

Lin et al (2006) Wang et al (1997), Barr and Hunt (1999), Teufel (1989) and Wolter and Berckhemer (1989) discovered that the nonlinear samples, in their

investigations, developed micro fractures after drilling. Acoustic emissions were monitored for some time after drilling and these emissions were related to an increase in strain. In addition, Teufel (1983, 1989) found that seismic wave velocities could be correlated to the amount of anelastic strain recovered in any particular direction. These phenomena were interpreted as being caused by the generation of micro cracks during anelastic strain recovery. The method is based on the assumption that over-coring of stressed rock can lead to differential stress and strain relaxation. An unloaded specimen, therefore, contains residual stresses that can result in the formation or opening of micro cracks. Relaxation of residual stresses may have a time-dependent component, according to which the rate of (anelastic) strain recovery decreases with time. Sakaguchi et al (2002) investigated the stress concentrations associated with the over-coring process and they concluded that the tensile stresses induced near the end of a core stub play a major role in micro-fracturing in a rock core during stress relief. This last is an important conclusion, as the over-coring process may cause (additional) damage in a core and promote the strain relaxation process.

While it is obvious that the composition of a rock will affect its relaxation response, no practically relevant information that related the degree of nonlinearity to composition was found in the available literature. The exact mechanism(s) for micro cracking upon relaxation has(ve) also not been determined.

In the petroleum industry, considerable interest in the quality of core and coring-induced rock damage has led to investigations into strain relaxation. Holt et al (2000) produced synthetic sandstones in order to simulate virgin conditions and to analyse the stress relaxation effects in a controlled manner. The synthetic sandstones were formed by allowing cementation to take place under stress. Holt et al (2000) argued that cementation under stress is representative of natural sandstone, and properties such as the Kaiser-effect, anelastic strain recovery and induced wave velocity anisotropy were observed with the synthetic sandstones. In the studies on synthetic sandstones, drilling effects could either be included or excluded by controlling the loading history.

The literature review and the observed presence of open cracks led the author to the conclusion that opening of micro fractures in response to stress relaxation can occur in Bushveld rocks. It is possible that the opening and/or generation of micro cracks is a direct response to the removal of the virgin *in situ* stresses and/or the induction of stress concentrations during overcoring of samples. The opening of such micro cracks is associated with excessive extension strains. It is thus highly likely that the magnitude of virgin *in situ* stress determines the onset of micro-fracturing. The nonlinear behaviour observed in samples drilled under relatively high stress conditions at Amandelbult and Union sites suggests that the tensile stresses induced near the drill tip as a result of the stress level in the rock mass may assist in the development of or even cause the opening of micro cracks. Samples retrieved from the Impala site, however, showed a nonlinear stress-strain relationship, even from areas destressed by mining. *In situ* stress change measurements at the site confirmed that fracturing or opening of existing fractures occurred as a result of stress relaxation (see Section 3.7.5). The laboratory test results from the Impala site are described in detail in sections 3.6 and 3.7 and the results from the other two sites are used for comparison. The following section deals with numerical models that were set up to compare the behaviour of linear elastic samples with open cracks to the observed nonlinear behaviour.

3.4 Modelling of crack behaviour

The observed nonlinear behaviour was reproduced numerically by introducing a single open fracture into a 2D elastic FLAC (Itasca, 1993) model. Quarter symmetry was used (Figure 3-2) to simulate repeating cracks. In the first model a quarter block of 30 mm x 30 mm with a crack of 11 mm length and 1 mm height (aperture) was simulated. This represents a crack density of 13%, which was calculated using the crack density parameter introduced by Bristow (1960). For a 2D geometry with a single crack this parameter is calculated by dividing the squared length of the crack (or sum of the crack lengths where there is more than one crack) by the area of the block ($11^2/30^2$) and expressing the result as a percentage. The block was loaded and the resultant stress-strain curve is shown in Figure 3-3. The dotted line represents the modulus of the uncracked material.

The crack closed at a high stress because of its large aperture. A smaller aperture would have resulted in a lower crack closure-stress.

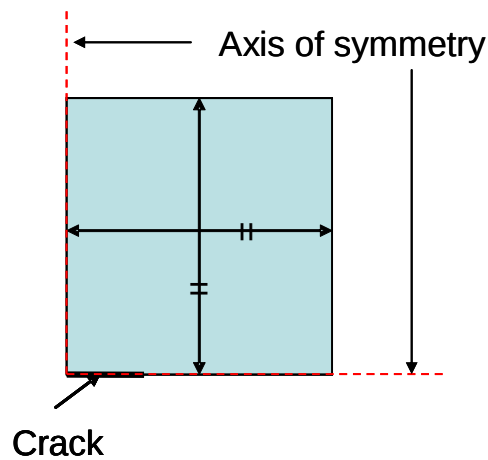


Figure 3-2 Diagram showing the quarter symmetry FLAC model used to simulate open micro fractures

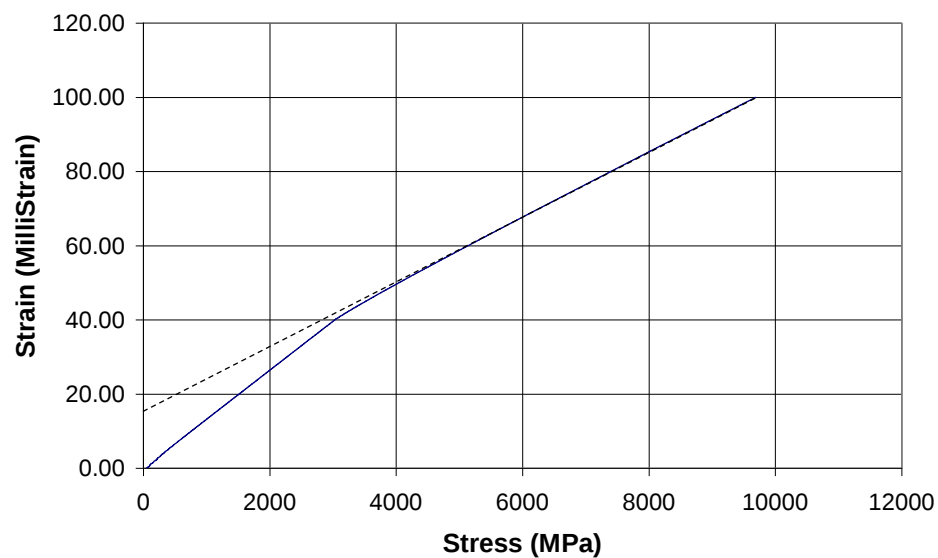


Figure 3-3 Results of the FLAC model for a 30 mm x 30 mm block with a crack of dimensions 1 mm/11 mm (aperture/length)

A second set of models was run on blocks of 15 mm x 15 mm and the same crack length (i.e. quadruple the crack density) and the crack aperture was varied. The results showed that quadrupling the density halved the modulus and the kink

indicating crack closure was dependent on crack aperture as expected (Figure 3-4).

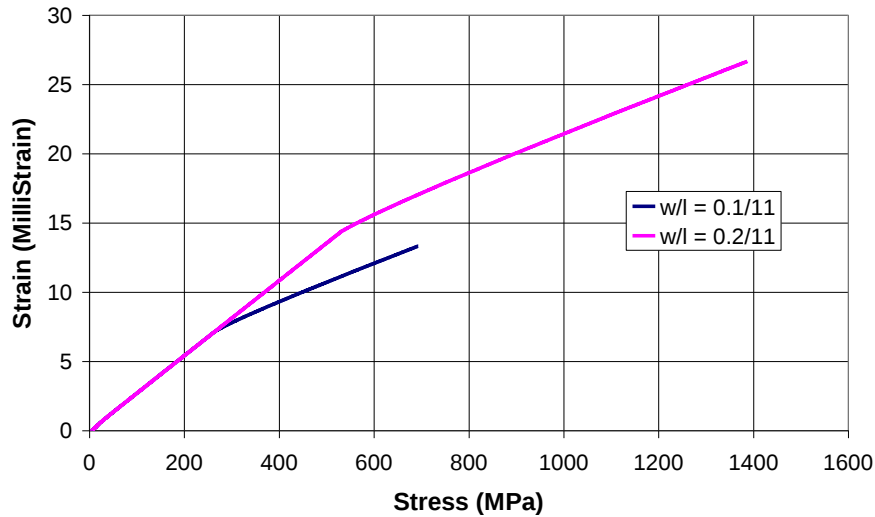


Figure 3-4 Results of the FLAC model for a 15 mm x 15 mm block with crack of dimensions 0.1 mm/11 mm and 0.2 mm/11 mm (aperture/length)

This model could not explain the significant lateral strains measured in the UCS tests (Figure 3-1). For this reason elastic DIGS models (Napier and Hildyard, 1992) with cracks were run to determine if the high lateral strains could be explained by shearing on appropriately oriented cracks. The models showed that shearing has little effect on dilation unless the shear cracks are associated with “wing-cracks”. A simplified shape of “wing-crack” was predetermined in the models, as shown in Figure 3-5, to simulate the effects of grain boundaries. Under uniaxial loading conditions, there was a linear response of crack opening and dilation to the applied stress if these cracks did not extend. However, if the dilating cracks become longer, irreversible damage occurred and the lateral response to stress was nonlinear. Most of the laboratory samples that showed a nonlinear axial strain response to uniaxial loading had an almost linear but relatively high lateral-strain response to this stress in the initial stages of loading as shown in Section 3.1. Nonlinear lateral strain was, however, observed in a few tests. Permanent damage was also suggested by the hysteresis on a biaxial sample that was cycled (Figure 3-6). Since most of the uniaxial and biaxial tests showed a linear response of lateral strain to the applied load in the early part of

the stress-strain curves, it appears that generally the “wing cracks” did not extend until the yield point had been reached. The model also showed that a linear relationship exists between stress and strain in the axial direction as a result of shearing but that the shearing acts to reduce the modulus further.

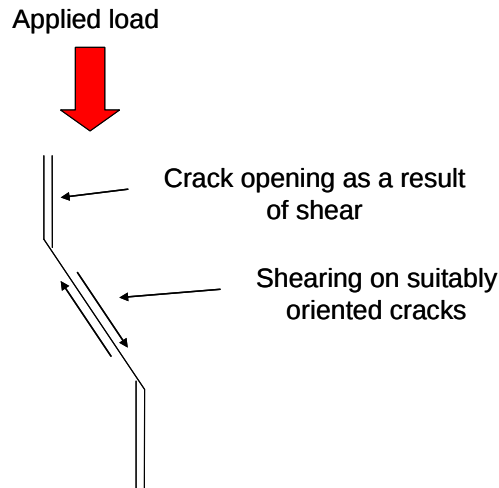


Figure 3-5 Lateral strain from shearing on micro cracks and opening of existing “wing-cracks” (around grain boundaries)

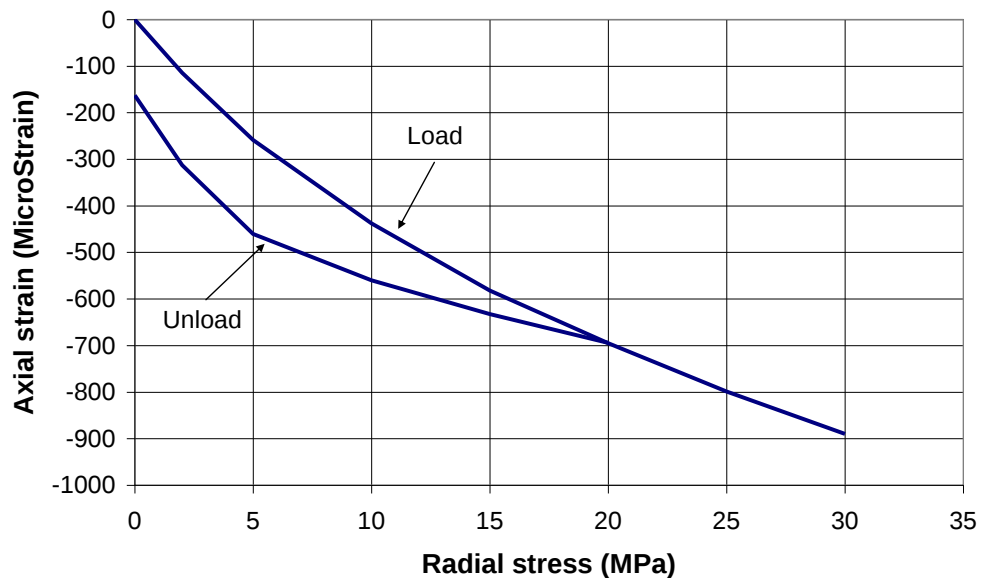


Figure 3-6 Strains in the unconfined direction of a laboratory biaxial test from the Impala site, showing hysteresis and permanent damage. The sample was loaded and unloaded around its circumference and the strains in the axial direction were monitored

As discussed in Section 3.3, Sakaguchi et al (2002) suggested that the tensile stress that develops at the drill tip during drilling could create or open micro cracks. Since this mechanism could explain why only some of the samples from the Amandelbult and Union sites were nonlinear, further investigations were carried out. The results of these investigations are described in the following section.

3.5 Stress conditions near the tip of a drill bit

Tensile stresses develop in the core at the end of a borehole if the rock being drilled is compressed. The tensile stress distribution in the core is dependent on the ratio of compressive stresses in the rock mass, oriented in the axial and radial directions of the borehole, and could vary as shown in Figure 3-7 (“A” and “B”) (Kaga et al, 2003). Elastic, axi-symmetric FLAC modelling shows a direct correlation between the magnitudes of the lateral compressive stress in the rock mass and the induced tensile stress in the core. In the case of zero axial stress (such as a vertical borehole drilled from the centre of a horizontal tabular stope), the induced tensile stress is approximately 15% of the radial stress.

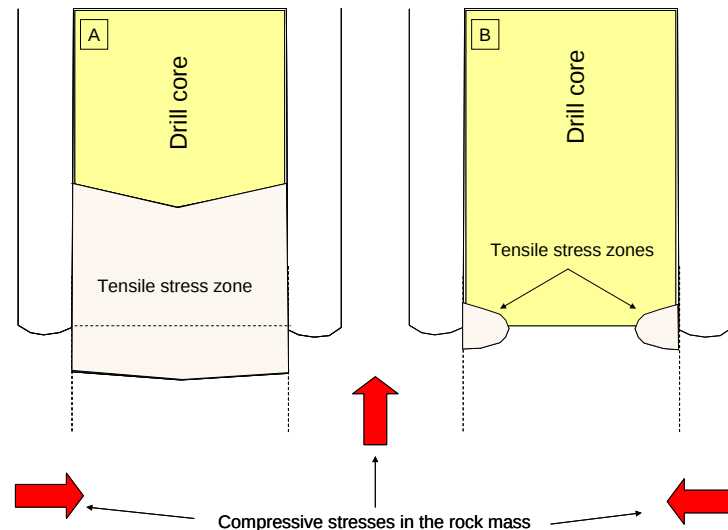


Figure 3-7 Schematic cross-sectional views of the tensile stress distributions in borehole cores drilled under low (A) and high (B) axial stress conditions (after Kaga et al, 2003)

Section 3.6 describes the nonlinear behaviour observed during UCS testing and compares the strain-stress results to linear elastic samples of the same rock type. An equation is formulated to describe the nonlinear behaviour, which also allows the nonlinear and linear contributions to be identified from the measured strain.

3.6 Characterisation of nonlinear behaviour

This section describes the nonlinear elastic behaviour and compares the results of UCS tests performed on nonlinear and linear elastic samples of the same rock type. Some creep tests are also discussed. Finally, an equation is suggested to describe the nonlinear behaviour.

3.6.1 Locations of the test samples

The samples described in this section were all retrieved from the Impala site. The orientations and positions of the boreholes used to provide the samples are shown in Figure 3-8. The dip of the strata is about 10° and the inclined boreholes were mostly about 15° from horizontal (5° steeper than the reef). Samples from the shallow-dipping boreholes were selected from above the stope and pillars. These samples were compared to samples from the vertical boreholes.

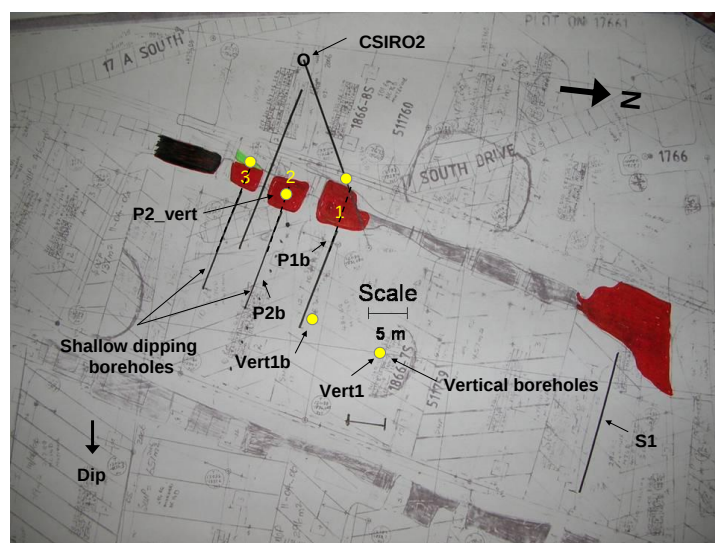


Figure 3-8 Impala site stope sheet: showing the positions of the boreholes used to retrieve the laboratory test samples

3.6.2 Strain-stress comparisons between uniaxial tests from shallow and deep elevations

An example of the nonlinear strain-stress behaviour of samples from the Impala site compared to the linear behaviour of the same footwall rock type at shallow depth is shown in Figure 3-9. Both cores were from Impala Platinum mines but the shallower-depth borehole was drilled from a haulage under virgin stress conditions and the deeper core was retrieved just ahead of the mining face, from Borehole P2_vert, shown in Figure 3-8. Figure 3-9 shows that not only are the tangential modulus and Poisson's ratio affected, but the material strength is also reduced. The weaker strength is suggestive of a less coherent, micro-cracked material and the nonlinear stress-strain relationship in the loading direction indicates that these micro cracks are open (Section 3.3). This theory, which relates the nonlinear behaviour to open micro cracks, agrees with the SEM findings of open and closed micro cracks in the nonlinear and linear samples respectively. If open micro cracks are assumed to be responsible for the nonlinear behaviour, the curved shape and the significantly lower magnitude of the tangential modulus of the nonlinear material in Figure 3-10 can be explained by crack closure and the initiation of permanent damage before all the cracks are closed. However, sliding cracks probably also contributed to the lower modulus.

Early damage could initiate on appropriately oriented, sliding cracks, as discussed in Section 3.3. Further confirmation of an early onset of failure is shown by the 1100 m curve shown in Figure 3-11. The relatively large lateral (radial) strain shown in Figure 3-9 is further evidence of sliding on unfavourably oriented micro cracks, which results in the opening of cracks oriented as shown in Figure 3-5 ("wing-cracks"). Similar behaviour could be expected from the deformation of soft inclusions or closure of open micro cracks. In these instances, new fractures could form in tension in the direction of loading (similar to an open stope) and result in inelastic deformation in the lateral (radial) direction. The formation of new fractures means that the process is irreversible or only partially reversible. In reality all the processes described might be taking place. However, no difference was observed in the mineral composition of the linear and nonlinear materials (Appendix A), thus the influence of this mineral composition can only be small.

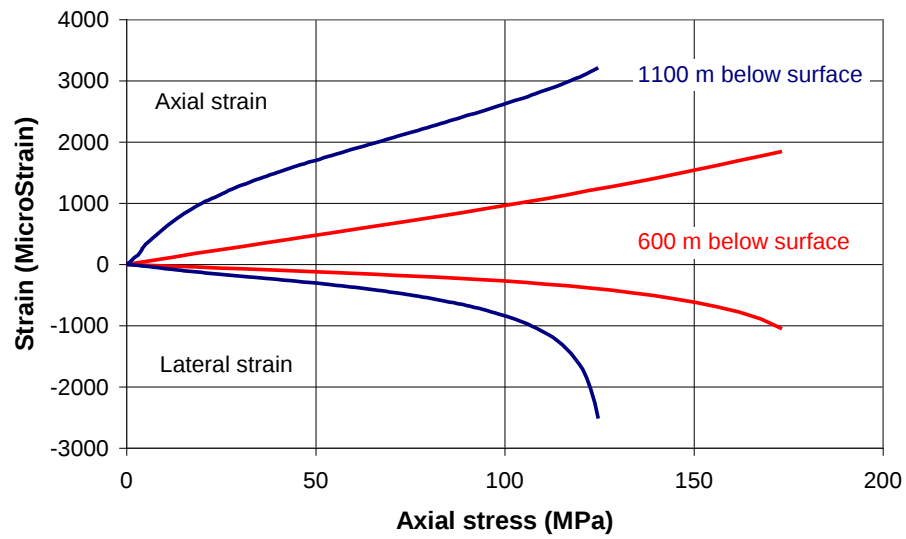


Figure 3-9 Strain-stress curves for spotted anorthosite from Impala Platinum Mine under uniaxial loading conditions. Cores retrieved at 600 m and 1100 m below surface. Upper curves: axial strain; lower curves: lateral strain

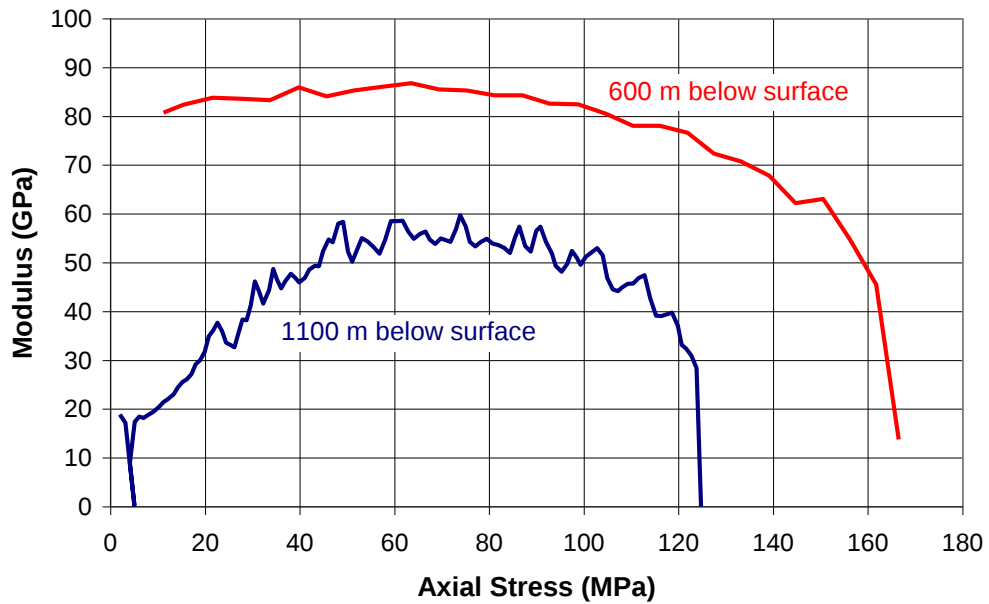


Figure 3-10 Comparison between the tangential modulus at 600 m and 1100 m below surface for anorthosite from Impala Platinum Mine under uniaxial loading conditions

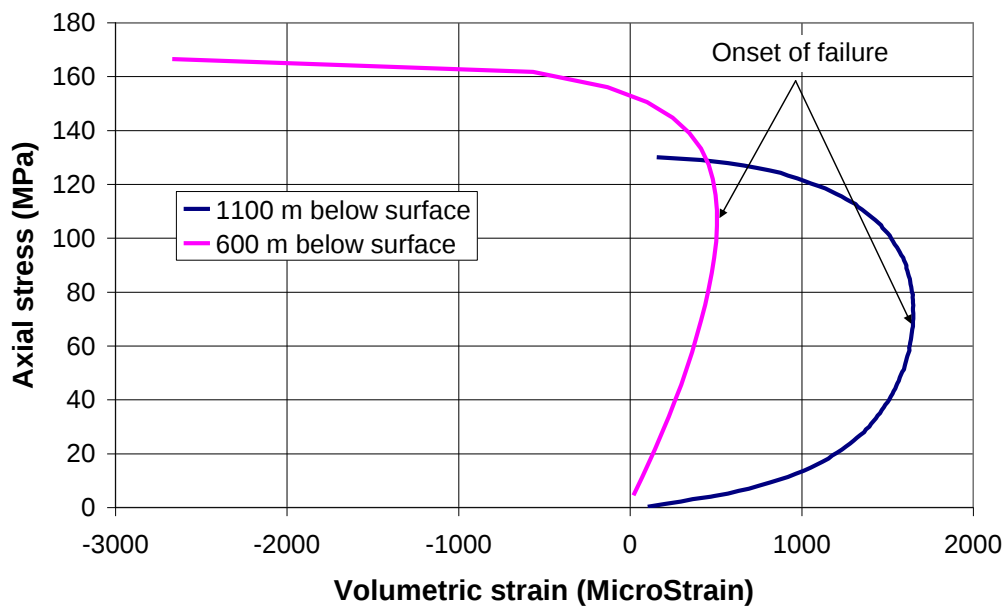


Figure 3-11 Volumetric strain-stress curves showing the onset of sample failure under uniaxial loading conditions. Sample from Impala Platinum Mine

3.6.3 Behaviour of core samples from the Impala site under uniaxial cyclic loading

The curves in Figure 3-12 show the applied stress versus the monitored axial and lateral strains during five cycles of uniaxial loading and unloading on a cylindrical sample of spotted anorthosite, over a time span of one to three hours. Initially, the unloading was performed at a much slower rate than the loading curve. However, in every cycle the valve controlling the return of the pressurised fluid (used in the loading mechanism of the machine) suddenly opened, dropping the load to zero rapidly. This part of the curve is not recorded in Figure 3-12. However, the starting position of every cycle is the strain at which the unloading curve reached zero stress in the previous cycle. The starting points of the cycles in the figure indicate that permanent, non-recoverable axial and lateral strain developed within each cycle. However, there is evidence (Hawkes et al, 1973) that this strain may be reversible with sufficient time. The apparent permanent strains affecting both the axial and radial (lateral) directions can be observed even at stress levels as low as 25% of the peak strength. Interestingly, the

apparent non-recoverable lateral strains are greater than the corresponding axial strains, particularly when the sample was cycled to higher stress levels ($\sim 150 \mu\epsilon$).

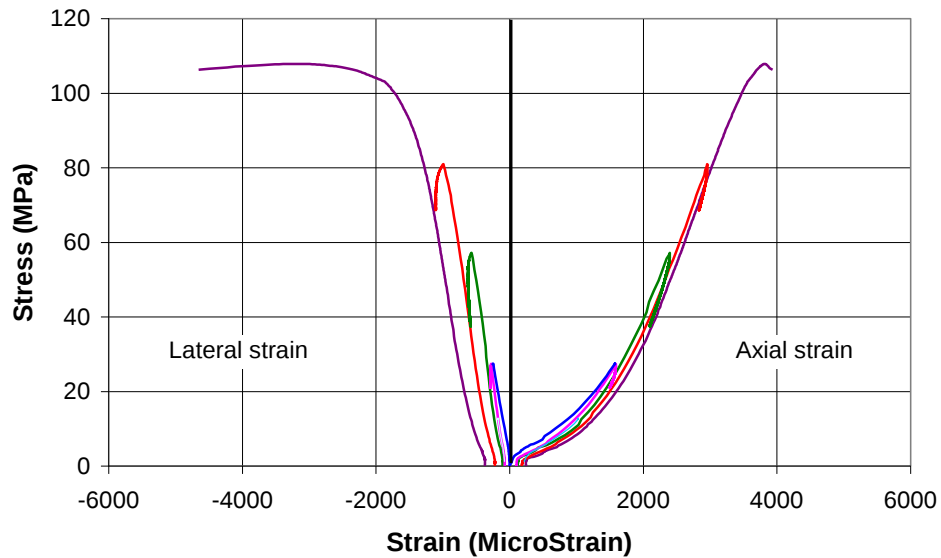


Figure 3-12 Cycles of loading followed by very slow unloading of anorthosite (soft, load-controlled machine)

The process of inelastic strain generation seems to be time dependent. This time dependency can be appreciated from Figure 3-12, where the lateral and, to a lesser degree, the axial strain continue to increase near the top of the unloading cycle. An explanation for this phenomenon, which was observed in all six samples that were cycled, is that the effect of the loading was not fully absorbed by the material at the onset of unloading – i.e. the standard loading rate was too fast and sliding on appropriately oriented micro fractures continued after loading had stopped; subsequent reversal in the direction of sliding occurred only after some time had elapsed. The possibility of creep was investigated using an MTS loading machine. At the time, the available equipment could only support triaxial tests. For this reason a relatively small confinement was used to provide conditions as close to uniaxial as possible. The strain increase during unloading was not observed when there was a three-hour time delay between the loading and unloading cycles (Figure 3-13). While it can be argued that a small confinement (1 MPa) could have affected the sample behaviour, the confinement should have restricted the axial (lateral) dilation and clamped some appropriately

oriented micro cracks and reduced the sliding of micro cracks. However, it can be clearly observed that creep occurred during the three hours when the load was maintained at the maximum load and no lagging of strain reversal took place when the sample was unloaded.

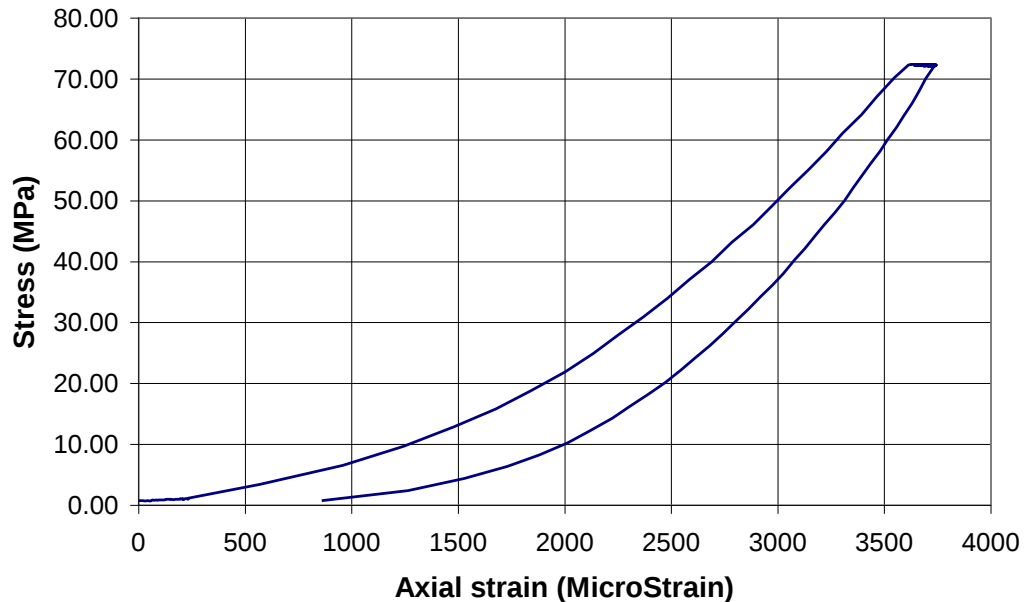


Figure 3-13 Cycle of loading and unloading of anorthosite with a delay of three hours between the cycles (MTS machine, triaxial test with 1 MPa confinement)

The lagging of the strain direction change when the loading was changed to unloading in the cyclic tests (Figure 3-12) can be explained by the concept of sliding cracks – i.e. the sliding did not keep up with the loading. Sliding crack theory also explains the observed hysteresis in these tests. In addition, the very steep region of the curves at the start of each loading cycle, as seen in Figure 3-12, suggests that the shearing on micro fractures had not fully reversed after the previous unloading cycle. Thus, at least some of the so-called “non-reversible” strain may have reversed if sufficient time had elapsed between the cycles. In essence, the behaviour of the Impala anorthosite can be described as “nonlinear elastic”, with only a small degree of largely recoverable hysteresis.

3.6.4 Creep in the rock samples from the Impala site

Creep effects are best quantified by maintaining a steady load over a long period of time. Figure 3-14 shows the result of a creep test on an anorthosite specimen that was subjected to an axial stress of 72 MPa and a confining stress of 1 MPa in the Wits MTS machine (same test as shown in Figure 3-13). The test lasted three hours and suggests significant creep (in both the axial and lateral directions) within the first half hour of loading. However, creep continued to take place for the full three hours at a decelerated rate (steady state creep).

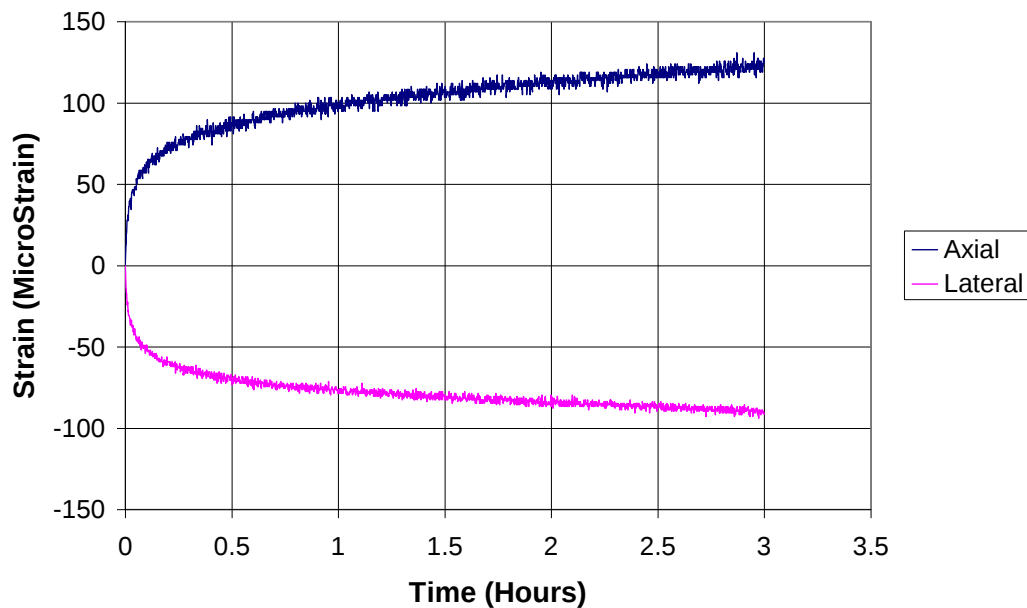


Figure 3-14 Creep test on anorthosite at axial stresses of 72 MPa and 1 MPa confinement

While the time-dependent strains seen in Figure 3-14 are relatively small compared to the total strain, they do seem to be associated with the nonlinear component of strain. It is of interest to note that both the axial (compressive) strain and the lateral (dilatational strain) are affected. At this stage of the research it is not clear if time-dependent effects are always associated with nonlinear behaviour. Neither can the observed creep in these UCS tests be used as an argument for anelastic strain recovery, i.e. the relaxation process during drilling, when the micro fractures are assumed to form. A better understanding of the mechanisms behind the nonlinear behaviour may have been achieved if strain

and acoustic measurements had been made soon after the cores were drilled. However, the required equipment was not available and the measurements were not done.

3.6.5 'Matrix' elastic constants

Triaxial tests were conducted on anorthosite at a confinement of 30 MPa to determine the elastic properties of the material when the micro cracks were closed – i.e. the 'matrix' elastic constants of unfractured rock. The graphs in Figure 3-15 and Figure 3-16 show the load and unload curves from hydrostatic conditions to an axial stress of about 140 MPa and back respectively. A noisy graph was produced, which is probably the result of using temporarily modified testing and monitoring equipment. In addition, the confinement was maintained manually by opening a valve, which resulted in slight variations in confinement during the test. The thick black lines seen in Figure 3-15 and Figure 3-16 were drawn between the loading and unloading curves. The 'matrix' elastic constants are assumed to be represented in the region where the lines are horizontal. A tangential modulus and Poisson's ratio of 83 GPa and 0.32, respectively, were thus assumed for the hangingwall anorthosite at the site. These constants compared favourably with the UCS tests conducted on linear anorthosite from the Amandelbult and Union sites. Of particular interest is the fact that the maximum tangential modulus of around 83 GPa is reached when the applied stress reaches a magnitude of approximately 110 MPa. If crack closure is the only reason for the nonlinear behaviour then this would suggest that complete crack closure only occurs at stress levels that are far in excess of the *in situ* virgin stresses.

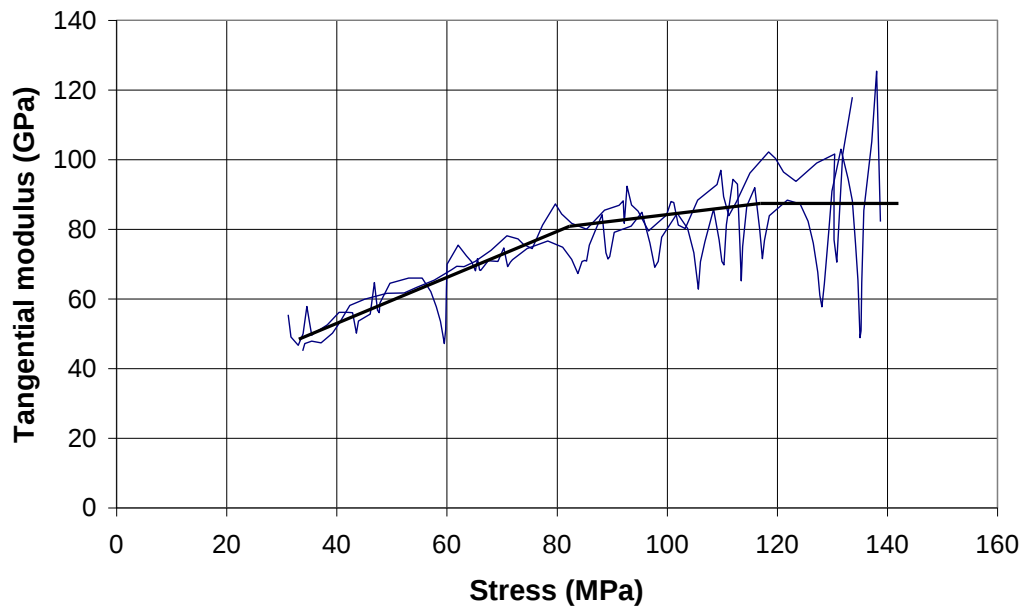


Figure 3-15 Tangential modulus as a function of axial stress, determined under triaxial loading and unloading conditions

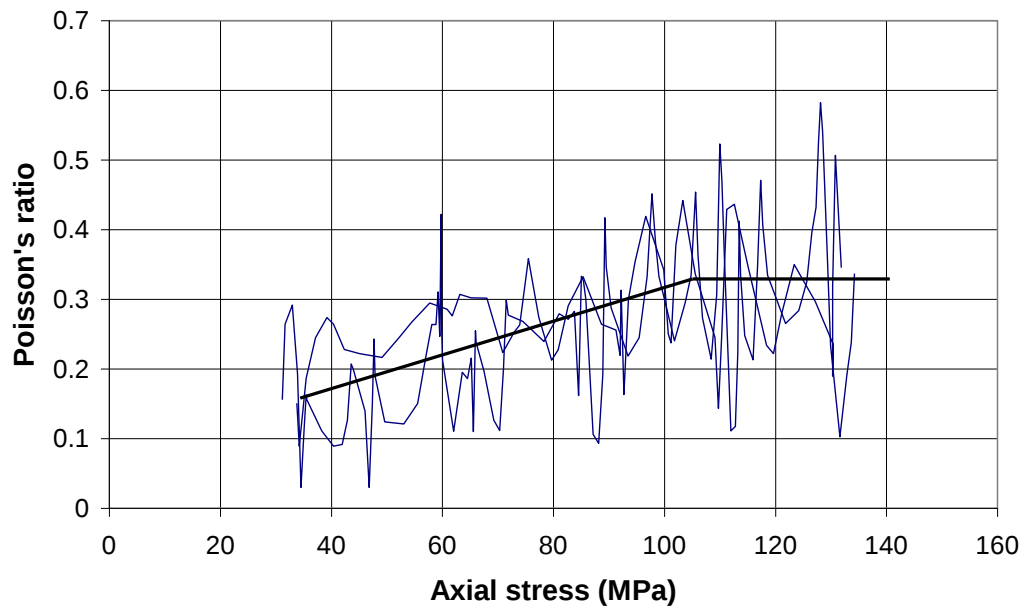


Figure 3-16 Tangential Poisson's ratio as a function of axial stress, determined under triaxial loading conditions

3.6.6 Equation to describe nonlinear behaviour

The nonlinear component of strain can be separated from the linear component with the use of Equation 3-1. This solution is based on the backfill ‘hyperbolic’ compaction of a porous material (Ryder and Jager, 2002), and is similar to the compression behaviour of a joint under normal loading (Goodman et al, 1968):

$$\varepsilon_a = \frac{\sigma_a}{E} + \frac{b\sigma_a}{a + \sigma_a} \quad 3-1$$

Where:

- ε_a = the nonlinear axial strain component
- σ_a = axial stress
- E = ‘matrix’ modulus
- b = maximum amount of nonlinear axial strain that can be generated (total crack closure)
- a = the stress at which half of the nonlinear strain ($b / 2$) is generated

By tuning the “ a ” and “ b ” values in Equation 3-1, a remarkably good fit can be obtained between the measured strains and the strains calculated on the basis of the equation 3-1. An example of this fit is shown in Figure 3-17. Note the unrealistically high Poisson’s ratio (0.9) that was used to simulate the lateral strain. It is suggestive of sliding cracks and opening of “wing cracks”.

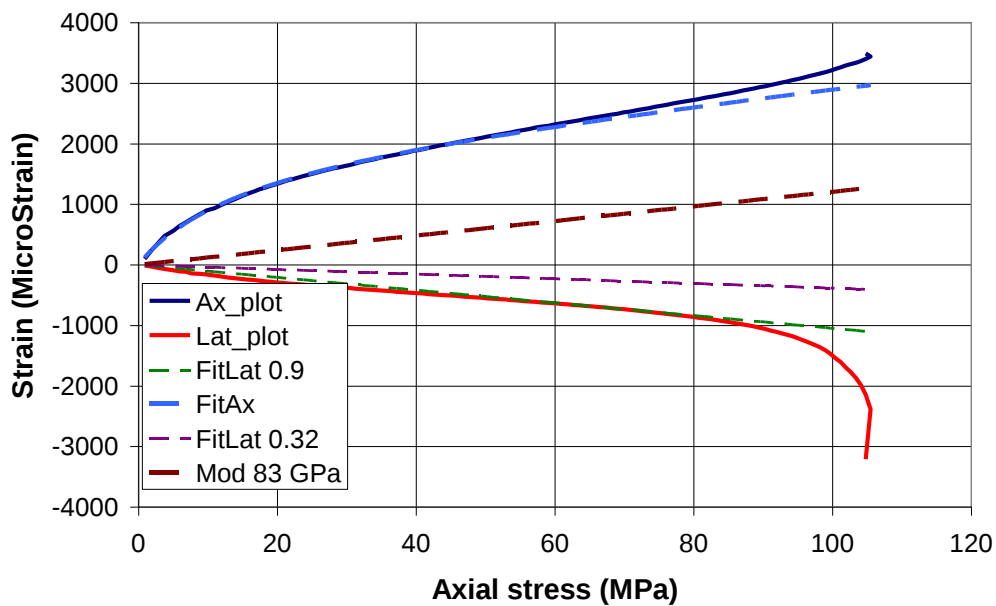


Figure 3-17 Strain-stress curves from a core extracted from a horizontal borehole (P1b in Figure 2) showing the fitted data determined from Equation 3-1 with $a = 15$ and $b = 1940$. The lateral plot was fitted assuming a Poisson's ratio of 0.9 and the plot for a Poisson's ratio of 0.32 is also included for comparison

The initially steeper radial (lateral) strain-stress curve shown in Figure 3-17 was not observed in all samples, but appears more commonly in samples from the vertical boreholes. The reduction in the slope of the lateral curve coincides with a reduction in the axial strain-stress curve from about 20 MPa in Figure 3-17. The reduction in the slope of the axial strain curve is explained by the closure of some of the micro cracks or a change in the underlying nonlinear mechanism; e.g. all or most of the open cracks closed at a stress of 20 MPa and the remaining nonlinear behaviour is a result of sliding on micro cracks. Closure of appropriately oriented sliding cracks would result in a higher friction along these surfaces and therefore a reduction in the slope of the lateral curve. The tangential modulus of the axial strain-stress curve never shows the 'matrix' modulus in a UCS test (Mod 83 GPa in Figure 3-17). This suggests that the onset of failure occurred before all the cracks were closed or sliding took place throughout the test.

The strain-stress result of a typical uniaxial test is shown together with the components of linear and nonlinear behaviour in Figure 3-18. The components were determined by back-fitting Equation 3-1 to the measurements. Again the

measured tangential modulus at every point along the curve in Figure 3-18 was less than the 'matrix' modulus (shown by the elastic component). A graphic description of the nonlinear component of stress in Figure 3-18 is provided in Figure 3-19. The tangent to the curve at zero stress cuts the “ b ”-strain asymptote at exactly half the asymptote strain – i.e. the strain associated with the “ a ”-value. The total axial strain, which is the measured strain, is simply the addition of the linear and nonlinear components of strain. Interestingly, it was discovered that the slope of the unload curves at the start of unloading always provides a good estimate of the 'matrix' modulus and Poisson's ratio (Figure 3-20). This was also observed by Walsh (1965).

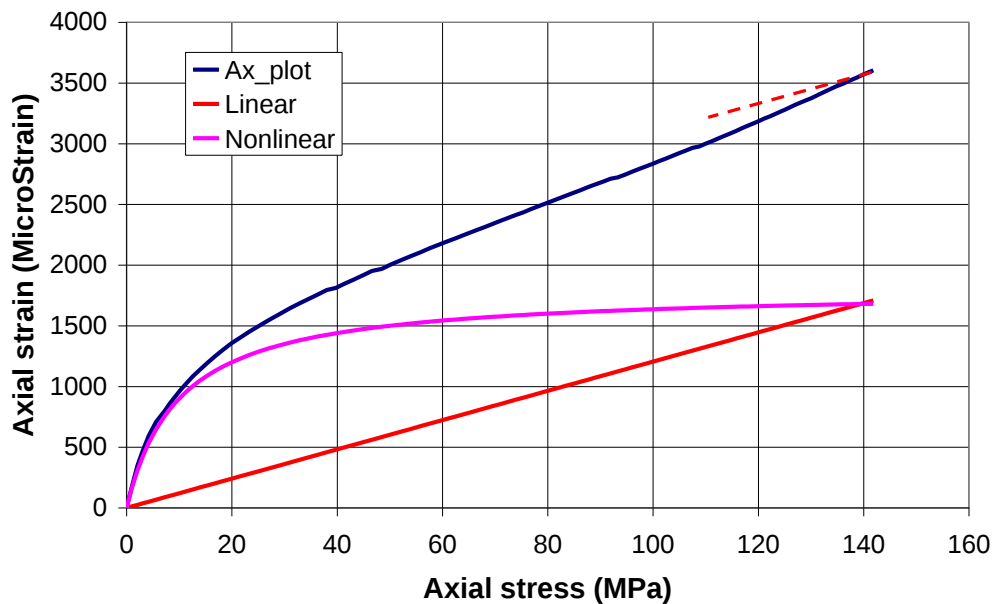


Figure 3-18 Typical nonlinear strain-stress behaviour of a uniaxial test and the separate components of linear and nonlinear strain within that test. The dotted line shows the deviation from the elastic response

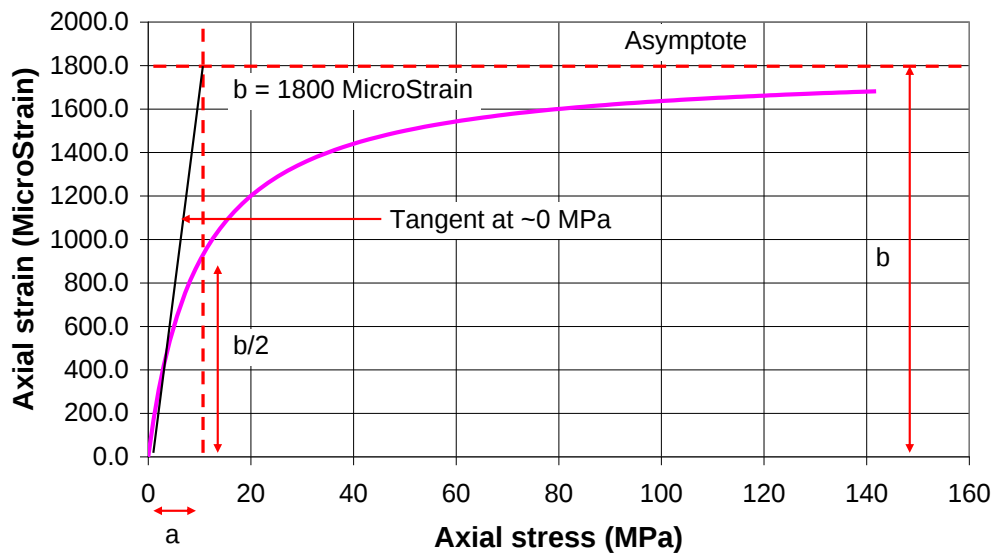


Figure 3-19 The nonlinear component of stress in Figure 3-18 is described by a 'hyperbolic' curve, and the interpretation of the "a" and "b" values is shown graphically

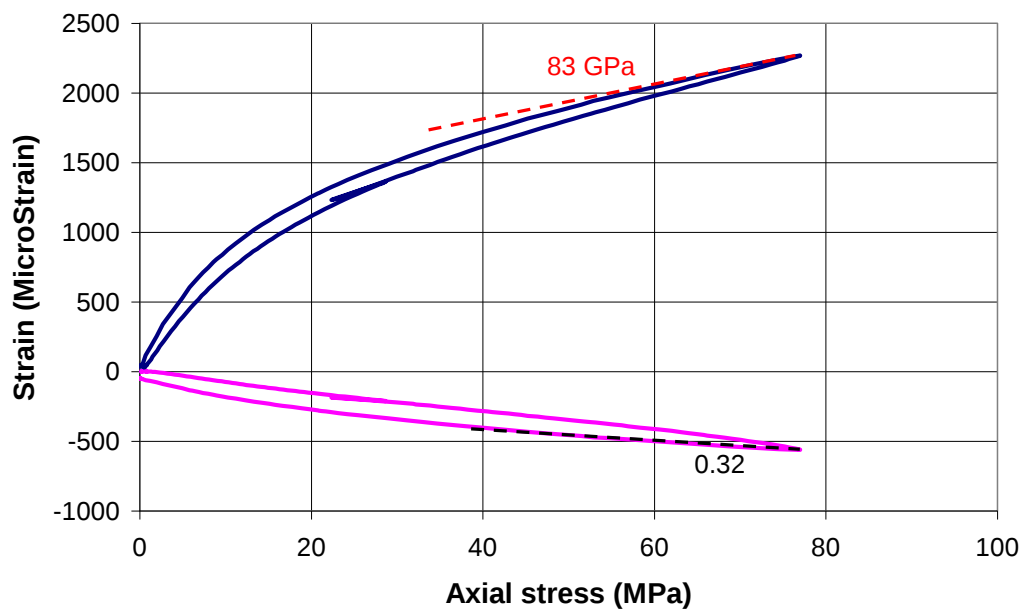


Figure 3-20 Loading and unloading of an anorthosite sample showing that the 'matrix' modulus and Poisson's ratio can be estimated from the start of the unload curves

3.6.7 Discussion

The observed nonlinear behaviour matches the behaviour of typical porous materials. All the tests and the literature suggest that the observed nonlinear behaviour is associated with open micro cracks. However, only a very limited number of high-magnification SEM investigations were conducted and further work is suggested to confirm the findings.

On the basis of available evidence, it can be argued that the onset of micro fracturing is directly related to the magnitude of the virgin *in situ* stresses. This stress relationship suggests that micro fracturing and opening of cracks occurs when sufficient differential strains develop between minerals with different moduli during relaxation. The argument assumes a state of equilibrium with any pre-existing micro cracks being healed in the virgin state. Once this state is modified by mining or drilling, the development or opening of micro fractures occurs if the virgin stress levels were high enough. It was observed that complete micro crack closure takes place only at stresses that seem to be far in excess of the (assumed) virgin *in situ* stresses. This can also be explained by the process of crack development or opening during relaxation because the exact virgin 3D stress condition was not applied in the tests. Even if the exact virgin stress conditions were achieved, broken asperities and miss-match of uneven surfaces (particularly if sliding takes place) would obstruct total closure. In addition, it is conceivable that adversely oriented cracks could have facilitated sliding even after the cracks approximately perpendicular to the direction of loading are closed. Open micro cracks can also explain the observed reduction in strength, as the presence of open micro cracks can be expected to have a negative affect on the material coherence.

Hysteresis and accompanying dilation, associated with cyclic loading and unloading, can be explained by sliding crack theory. Observations, therefore, suggest that the micro fractures are not only subject to opening and closing, but also to sliding. A creep test showed some time-dependent behaviour at compressive stresses that were well below the average strength of the tested specimens. While these results demonstrate the potential for time dependency, they do not prove that anelastic (time-dependent) strain recovery takes place

during unloading. This issue needs to be investigated using acoustic monitoring techniques in conjunction with strain change measurements over time. As only one creep test was conducted over three hours, further testing should be done to determine the long-term effects of creep.

Very high Poisson's ratios were determined at 50% of the UCS in the nonlinear samples. The high lateral strains can be explained by the effects of shearing on appropriately oriented cracks and opening of "wing-cracks". It was discovered that the 'matrix' modulus and Poisson's ratio can be identified at the start of an unload cycle, i.e. after loading a sample. This phenomenon can be explained if it is assumed that most of the micro cracks closed at a relatively low axial stress (probably approximately equivalent to the virgin stress condition), and that the lower modulus and nonlinear behaviour at higher stresses are a result of sliding on appropriately oriented, closed micro cracks. When the loading condition changes from static to unload, sliding on the cracks initiate only once sufficient strain has occurred to overcome the frictional effects on the micro crack surface (Kuijpers, 1998). Thus the initial strain is unaffected by the micro cracks in either the axial or lateral directions and is representative of the 'matrix' modulus and Poisson's ratio (Walsh, 1965).

The research suggests that the behaviour of nonlinear rocks is influenced by two components of strain: the solid rock around the cracks ('matrix') and the closure and sliding of micro cracks. It was therefore possible to separate out the effects of the cracks and describe the rock behaviour in terms of linear elastic constants and the behaviour of the cracks. An equation was derived from the backfill 'hyperbolic' for compaction of a porous material (Ryder and Jager, 2002), and the compression behaviour of a joint under normal loading (Goodman et al, 1968).

Section 3.7 describes the tests that were conducted to establish the mechanisms causing the nonlinear behaviour. In particular, it needed to be established why all the samples from the Impala site were non-linear and only some of the samples from the other two sites were affected by micro-fracturing. Possible influences of this nonlinear behaviour on the rock mass are also discussed in Section 3.7.

3.7 Mechanisms of micro fracturing at the Impala site

This section deals with specialised laboratory tests that were conducted to verify the theory developed in Section 3.5. It was important to establish whether the nonlinear behaviour was a product of drilling conditions only or if the rock mass around excavations was affected. The underground geotechnical conditions necessary for the development of nonlinear behaviour were also investigated.

3.7.1 Introduction

Several different laboratory tests were conducted on cylindrical cores of nonlinear rocks to establish:

- the effects of drilling on fracture development;
- the effects of shearing of micro cracks on sample behaviour; and
- if there is a preferred orientation of micro fractures.

Where possible, several different tests were conducted on a single rock sample to eliminate variation between samples. Cores from vertical and horizontal boreholes were used to establish preferred fracture orientation and to determine the underlying mechanisms for the observed nonlinear behaviour. Hydrostatic loading conditions were applied to reduce or eliminate the shear stresses. Biaxial loading conditions were applied across the core axis (lateral direction) to simulate the approximate stress condition of the rock retrieved from a vertical borehole in the hangingwall of the shallow-dipping stope.

Small-diameter samples were drilled across the diameter of larger cores, retrieved from both sub-horizontal and vertical boreholes, to determine the effects of drilling on the micro cracks. As the smaller orthogonal cores (Figure 3-21) were drilled in a stress-free environment, these samples were assumed to be free of drill-induced micro fractures. The results were compared to tests performed on the original core immediately adjacent to the orthogonal cores. A sketch showing the concept is provided in Figure 3-21. Both samples were tested uniaxially and,

therefore, the smaller sample was loaded orthogonally with respect to the original samples.

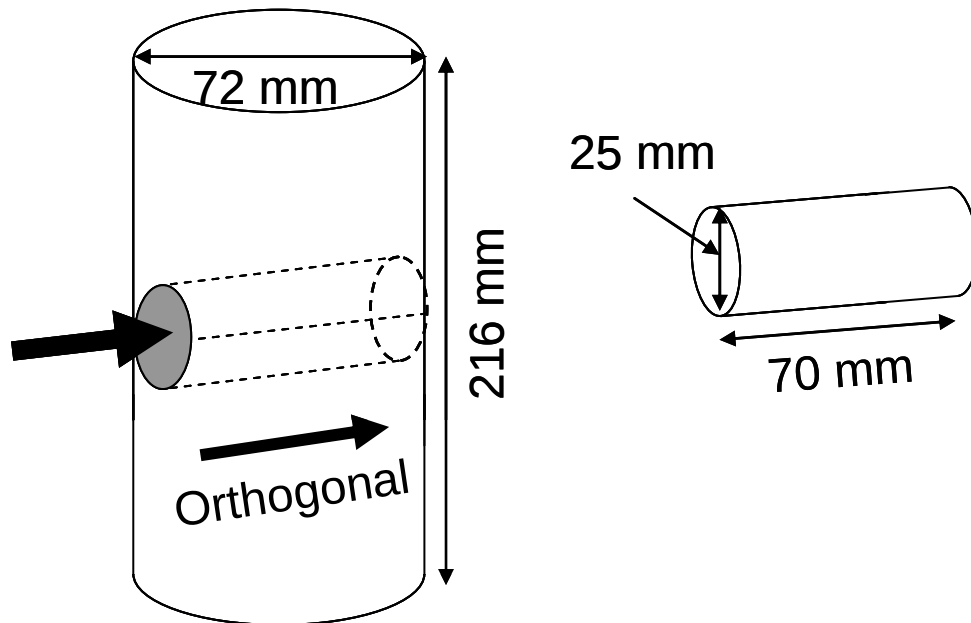


Figure 3-21 Sketch showing the orientation of the orthogonal cores

Finally, stress change measurements conducted in the stope hangingwall indicated that the nonlinear behaviour was not restricted to laboratory samples, but that the rock mass was affected as well. These measurements also indicated the conditions under which the nonlinear behaviour occurs in the rock mass. A small database of rock tests showed that the nonlinear behaviour probably initiates at a depth of about 1000 m below surface at Impala Platinum. The results of the database suggested that nonlinear behaviour probably requires a minimum virgin stress condition or depth below surface for the micro fracturing to initiate.

3.7.2 Hydrostatic loading

Figure 3-22 shows examples of hydrostatic test results that were conducted on cores from a vertical and a horizontal borehole in the hangingwall (Vert1b and HorizS1 in Figure 3-8 respectively; the test configuration is shown in Figure 3-23). It can be observed that, in the case of the vertical hole, the axial strains are much larger than the lateral strains. The core from the horizontal hole, drilled over an

open stope under stress relieved conditions, does not show a significant difference between axial and lateral strains. This observation of larger axial strains seems to confirm the hypothesis of drilling-enhanced nonlinear behaviour by opening micro cracks oriented in the lateral direction. The fact that this difference in strain is only observed in the vertical hole and not in the horizontal hole can be explained as follows. The horizontal borehole was located above a mined-out stope at the time of over-coring. This implies that relatively small lateral stresses and relatively large axial stresses would be present around this borehole. Such a combination of stresses is not conducive to the formation of drilling-induced tensile stresses ("B" in Figure 3-7) and the associated enhancement of micro fracturing. The core from the vertical borehole, on the other hand, was subjected to relatively large radial stresses (combination of both lateral directions) and a very low axial stress ("A" in Figure 3-7). The induced tensile stresses due to drilling are parallel to the axis of the core and therefore more likely to open lateral cracks. As the strains in the axial and lateral directions are the same for sample HorizS1, it appears that the drilling process had little or no effect on the behaviour of this sample. For this reason, the HorizS1 results, and the good correlation between the lateral curve of Vert1b and the HorizS1 curves suggest a level of fracturing that is not affected by the drilling process and probably existed in the rock mass prior to drilling. Further evidence of the nonlinear material behaviour and the validity of the 'matrix' constants was provided by an excellent match of Equation 3-1 to the data in Figure 3-22 using the 'matrix' constants and relevant 'a' and 'b' values (about 20 MPa and 1600 Micro Strains respectively). The curved shapes of all the strain-stress relationships under the hydrostatic loading conditions shown in Figure 3-22 are indicative of open micro cracks.

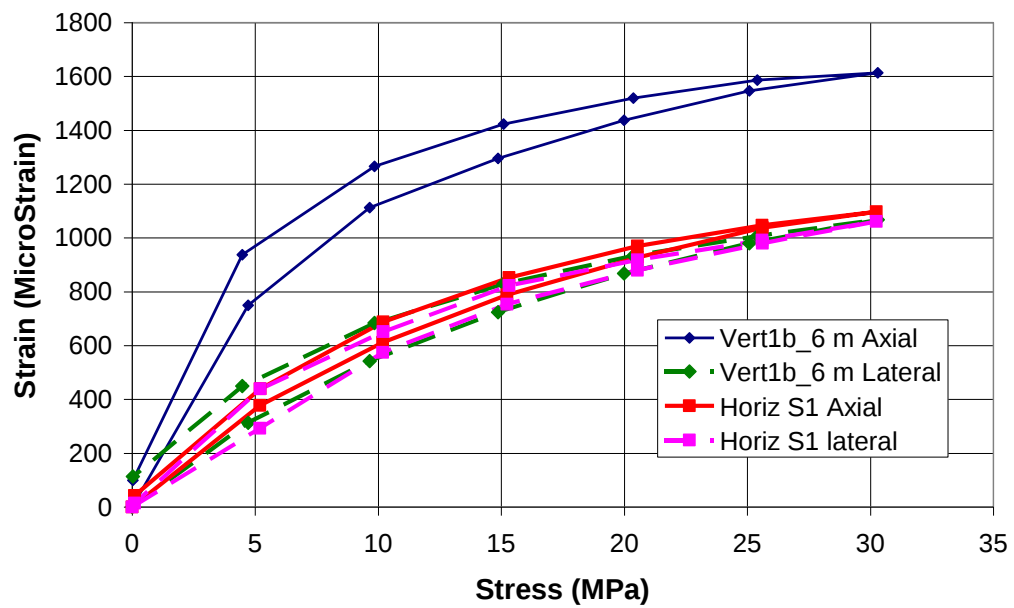


Figure 3-22 Strain-stress curves for hydrostatic tests performed on samples extracted under relatively higher (Vert1b – Figure 3.2) and lower (horiz S1 – Figure 3.2) in situ stress conditions

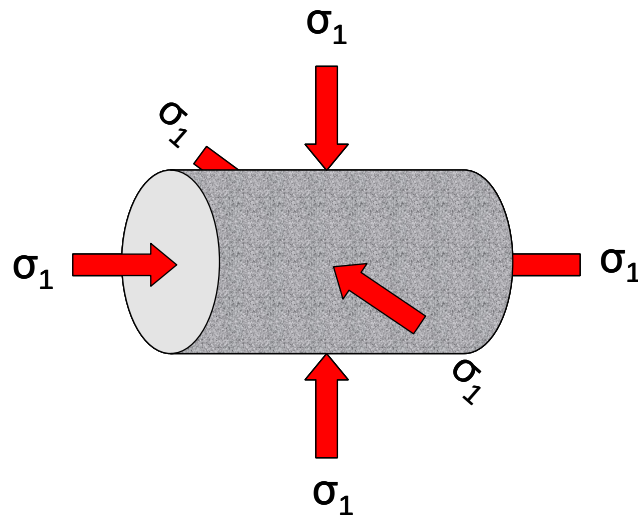


Figure 3-23 Diagram showing the hydrostatic test configuration

3.7.3 Biaxial loading

Figure 3-24 shows the results of biaxial tests that were conducted on a vertical and horizontal core, from the same boreholes as the hydrostatic tests (loading configuration shown in Figure 3-25). These tests show a similar nonlinear response in the lateral loading direction to that observed in the hydrostatic tests. The strain-stress behaviour in the lateral direction of the two samples was also similar, which could be expected if the drilling process only affects the micro cracks oriented in the lateral direction – i.e. perpendicular to the core axis. However, of interest here is the dilation in the unconfined axial direction. Relatively large axial dilation is associated again with the core from the vertical hangingwall borehole, which was drilled under relatively high radial (lateral) stress conditions. The large axial dilation in the core from the vertical hole is associated with pronounced hysteresis in response to the loading and unloading cycle. Some form of sliding and dissipation of (frictional) energy is assumed to be causing this response. The sliding cracks in turn will induce opening of cracks as shown in Figure 3-5, resulting in dilation, which is additional to the Poisson effect. Although less axial dilation was measured in the core from the horizontal hangingwall borehole (in the same material), the dilation is still greater than the Poisson effect (Figure 3-24). Thus crack sliding appears to have occurred in both samples, again suggesting a level of fracturing that probably existed in the rock mass prior to drilling.

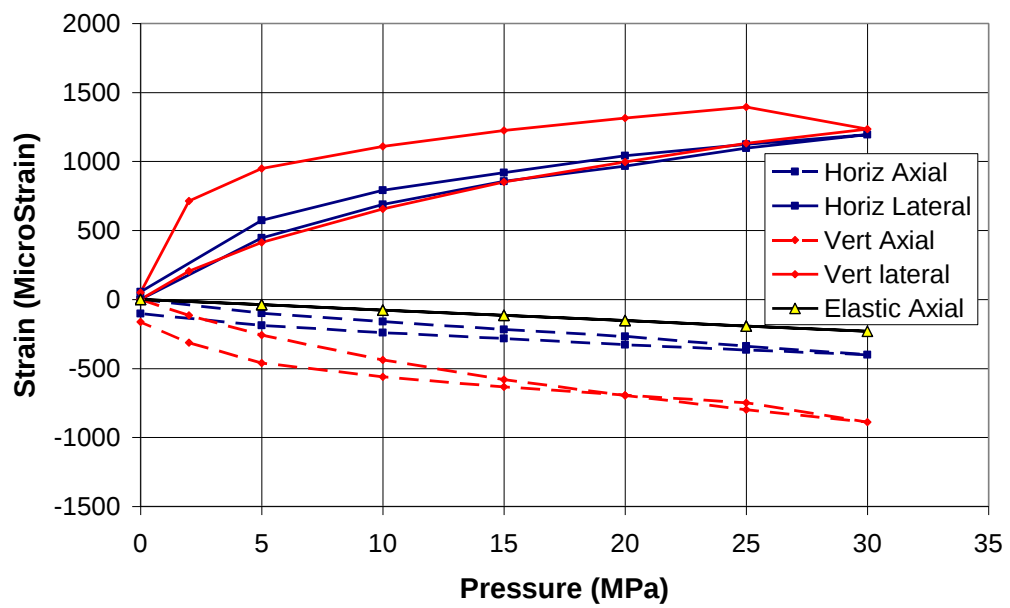


Figure 3-24 Stress-strain results under lateral biaxial loading conditions from cores retrieved from vertical (Vert1b_4.83 m) and horizontal (Horiz S1) boreholes

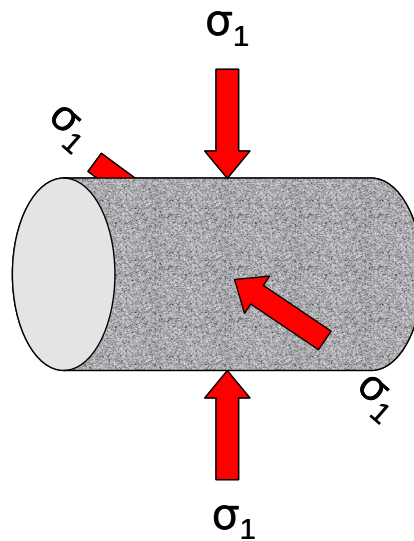


Figure 3-25 Diagram showing the biaxial test configuration

3.7.4 Core specimens drilled across the diameter of the original borehole cores

Uniaxial tests were conducted on smaller cores that were drilled across the original borehole cores (orthogonal tests) – i.e. in a direction perpendicular to the original borehole axes (Figure 3-21). The aim of the tests was to investigate the dominant direction of micro-fracturing and the effects of the original drilling, under the influence of the underground field stresses, on micro-fracturing. The original cores were drilled under relatively high stress and the smaller cores were subsequently drilled under destressed conditions. Therefore, no additional damage (drill induced) is expected in the smaller cores. Samples were selected from a vertical borehole drilled at the centre of Panel 7s (Vert1b), a horizontal borehole drilled over the top of a pillar (Horiz P2b) and a horizontal borehole drilled over the panel (Horiz S1). The borehole locations are shown in Figure 3-8. The results of the orthogonal tests are shown in Figure 3-26 and the results of uniaxial tests, which were conducted on the original borehole core adjacent to the orthogonal tests, are shown in Figure 3-27.

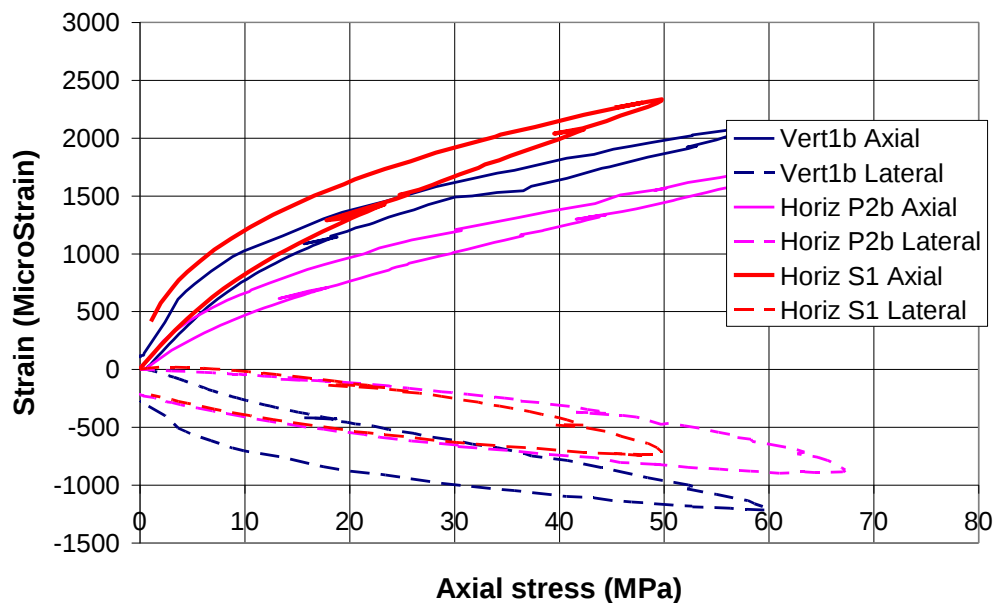


Figure 3-26 Results from UCS tests on small cores drilled across the original borehole cores from Vert1b and Horiz P2b (Figure 3.2)

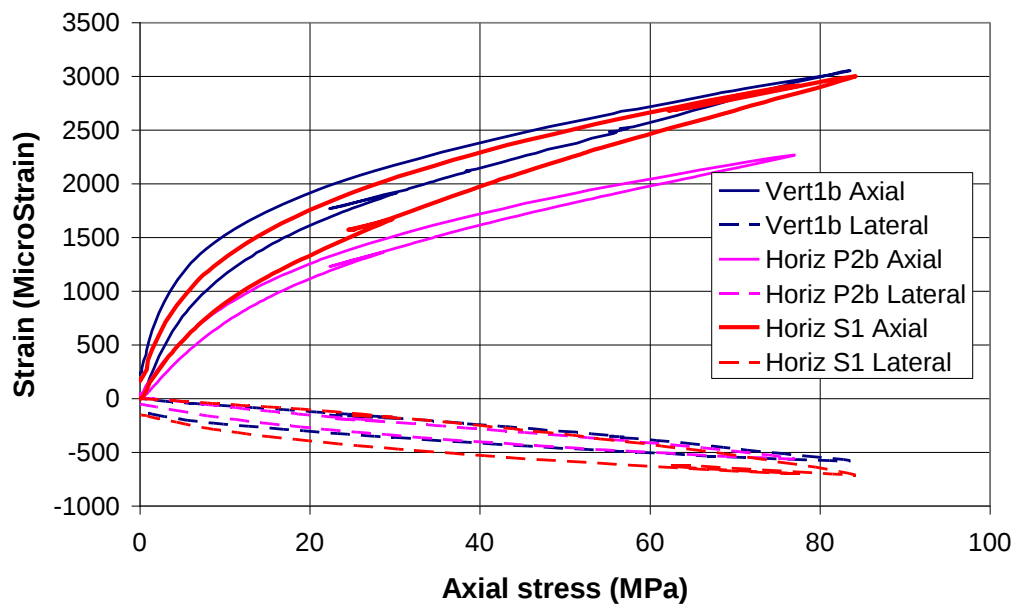


Figure 3-27 Result from UCS tests on original cores

The results shown in Figure 3-26 suggest variations in the distributions of micro fracturing and a dominant direction could not be established from the few tests. However, a comparison between Figure 3-26 and Figure 3-27 shows that the small, lateral cores exhibited smaller axial strains than the original samples. For this reason the cores that were drilled across the original cores were stiffer than the original cores. This would imply that the nonlinear component of strain is larger along the axis of the core irrespective of the direction of the borehole from which the cores are obtained. This conclusion is based on the assumption that the comparison is done on equivalent cores. It is, however, likely that variation in micro-crack distribution affects the response of individual cores. In addition, there could be local variations within a single core specimen. Smaller samples are thus more vulnerable to variation than larger samples. As these tests were conducted on a very limited number of specimens, any conclusions based on comparing results from different cores need to be treated with caution. Testing the same sample in different ways – such as was done in the hydrostatic, biaxial and uniaxial loading – provides more reliable information.

3.7.5 Underground instrumentation at the Impala site

If it is assumed that micro fractures open as a result of stress relaxation, it can be argued that the immediate circumference and blind end of a borehole will be micro fractured (Figure 3-28). In this way any gauges glued to a borehole for the purposes of stress measurements will be fixed to micro-fractured rock.

An axi-symmetric linear-elastic Elfen (2008) model with a Young's modulus of 83 GPa was set up with a comparatively low modulus (15 GPa) thin skin around the edge and at the end of a borehole (Figure 3-28). A Poisson's ratio of 0.23 was assumed in both materials. The low-modulus material was 17 elements thick, which is the equivalent of 0.85 mm in a borehole diameter of 32 mm. The aim of the modelling was to establish the effects of a thin layer of micro-fractured material on stress change measurements. The strain results of the low- and high-modulus materials are similar if compared to the applied stress (Figure 3-29). However, the stress in the locally micro-fractured rock is significantly lower than the stress in the surrounding unfractured rock mass. The applied stresses were a constant 30 MPa in the radial direction and the axial direction was varied as shown in the graph. The analytical solution for strain in linear elastic materials for the two moduli is also shown for comparison.

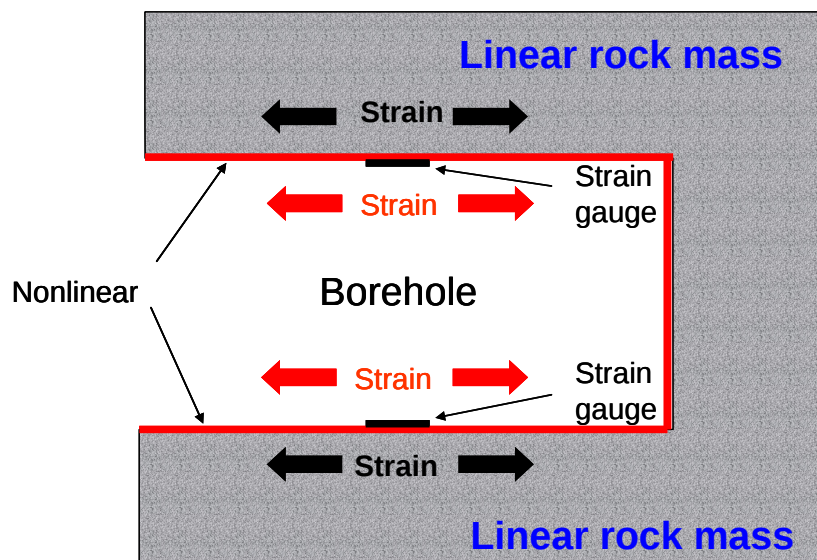


Figure 3-28 Diagram illustrating the concept of a thin skin of low-modulus rock around a borehole located in a higher-modulus linear elastic rock mass

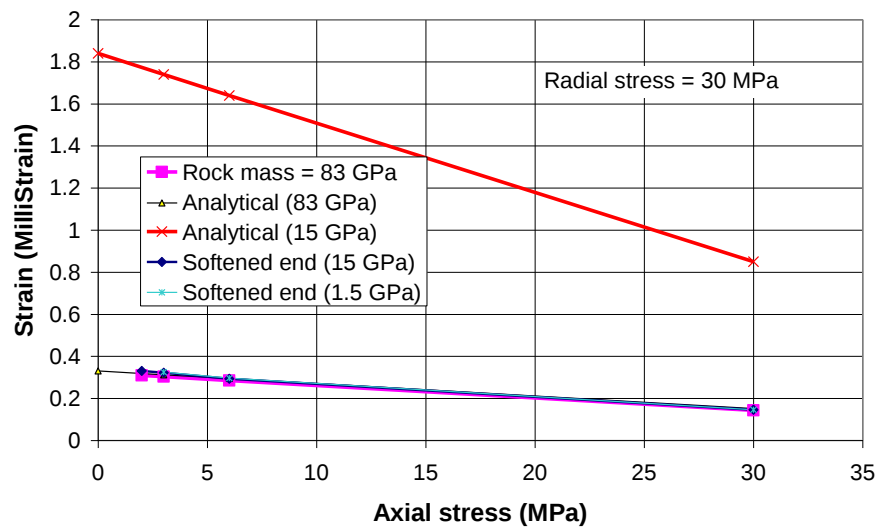


Figure 3-29 The stress-strain relationship around the circumference of a borehole in a linear elastic material with and without a thin skin of low-modulus material around the edge of the borehole. The model results are compared to the analytical solution for the two moduli

Since the measurements are strain-based, the stress in the locally softened material does not affect the measurements. Changes in stress can, therefore, be determined by applying the 'matrix' elastic constants to the measured strains, even if the rock mass is locally micro fractured.

A 3D straincell was installed about 5 m above a panel and ahead of the face (CSIRO2 in Figure 3-8). The stress change measured in a vertical direction is compared to an elastic MinSim (COMRO, 1981) model in Figure 3-30. The good correlation between the measurements and the elastic model up to and just beyond peak stress suggests that the concept of using the 'matrix' elastic constants to evaluate the stress-change measurements up to a face advance of about 5 m ahead of the cell was correct. However, further face advances were associated with an inferred stress (strain) change much greater than suggested by MinSim.

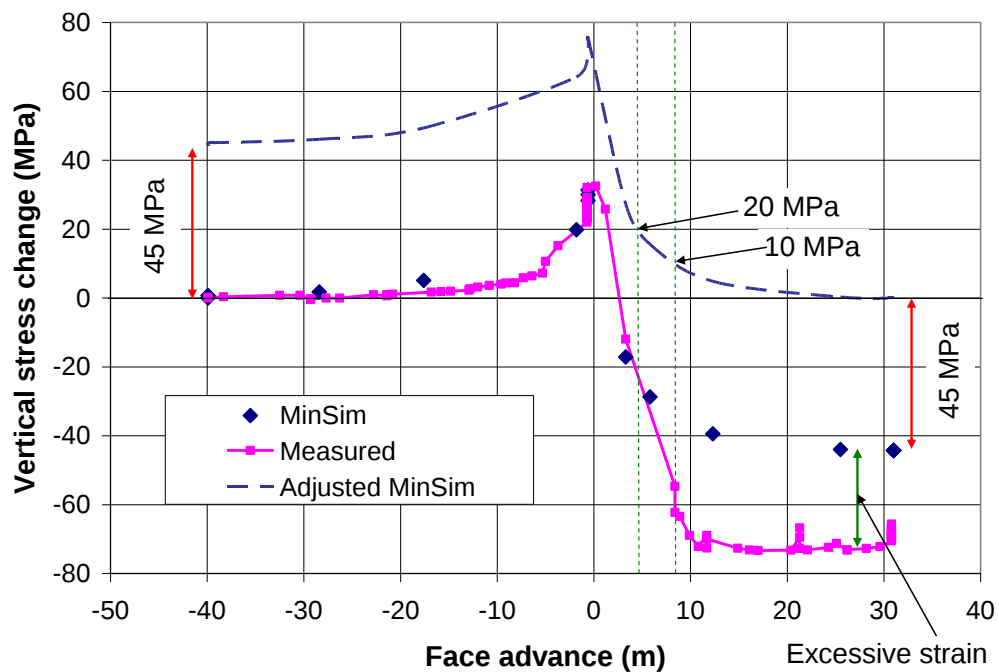


Figure 3-30 Impala site: change in vertical stress 4.7 m above the 30 m-wide 8s panel in response to undermining CSIRO2 (Figure 3-8) compared to an elastic (MinSim) model. Negative face advance refers to the distance of the cell ahead of the face. Dashed curve represents an upwards adjustment of the MinSim model by 45 MPa to show the field stress

The vertical stress at 4.7 m above the centre of a 30 m-wide, shallow-dipping panel when the face had advanced about 30 m ahead of the instrument was almost zero. For this reason the elastic stress change curve determined by MinSim can be adjusted upwards by about 45 MPa to show the field stress (Figure 3-30). From a face advance of about 5 m, the measurements suggest a much greater stress change than the elastic model indicating one of the following:

- the actual peak stress is much higher than predicted by the model, if the vertical stress measurements are adjusted by about 72 MPa to ensure almost zero stress conditions in the immediate hangingwall of the panel at the final face position;
- a significant tensile stress develops in the vertical direction in the immediate hangingwall if both curves in Figure 3-30 are adjusted upwards by 45 MPa, i.e. $-72 \text{ MPa} + 45 \text{ MPa} = -27 \text{ MPa}$; or
- additional strain was measured when the rock mass became nonlinear.

The most plausible explanation for the additional measured strain is that micro cracks opened when the strain differential between various crystals was large enough to break certain bonds. Thus the additional strain was measured when there was a change from linear to nonlinear elastic. The open-crack theory is substantiated by the SEM findings.

A comparison between the MinSim and measured profiles shows that the measurements deviated from the elastic model at a face advance of between 5 m and 8 m (Figure 3-30). The elastic model suggests that the vertical field stress was between 10 MPa and 20 MPa at this face advance. Thus the rock mass became nonlinear when the vertical stress dropped to about these magnitudes.

Further evidence of a nonlinear rock mass was shown by the stress change measurements conducted between 3.2 m and 5.3 m above the three highlighted pillars shown in Figure 3-8. One of these measurements is shown in Figure 3-31 as an example of a rapid drop in stress to inferred “tensile”. As a tensile stress above a pillar is impossible, even if the pillar is in its residual state, the measurements appear to show additional strain to that expected by a simple stress drop. This additional strain may be due to the development of a nonlinear rock mass around the borehole. In this instance the stress drop was associated with a seismic event, which could have enhanced the micro-fracturing process in the rock mass around the borehole. The sudden inferred stress drop appears in most cases to have occurred at a vertical stress of about 10 MPa. In essence, a relaxation process appears to be initiated and micro fractures opened when the stress conditions drop sufficiently. The associated monitored strains cannot be used to infer real external stress changes. These observations have demonstrated that micro damage and associated nonlinearity (release of the “*b*-component” of strain) can take place in response to the removal of *in situ* stresses. At the Impala site this appears to have occurred once the stresses dropped below about 10 MPa to 20 MPa.

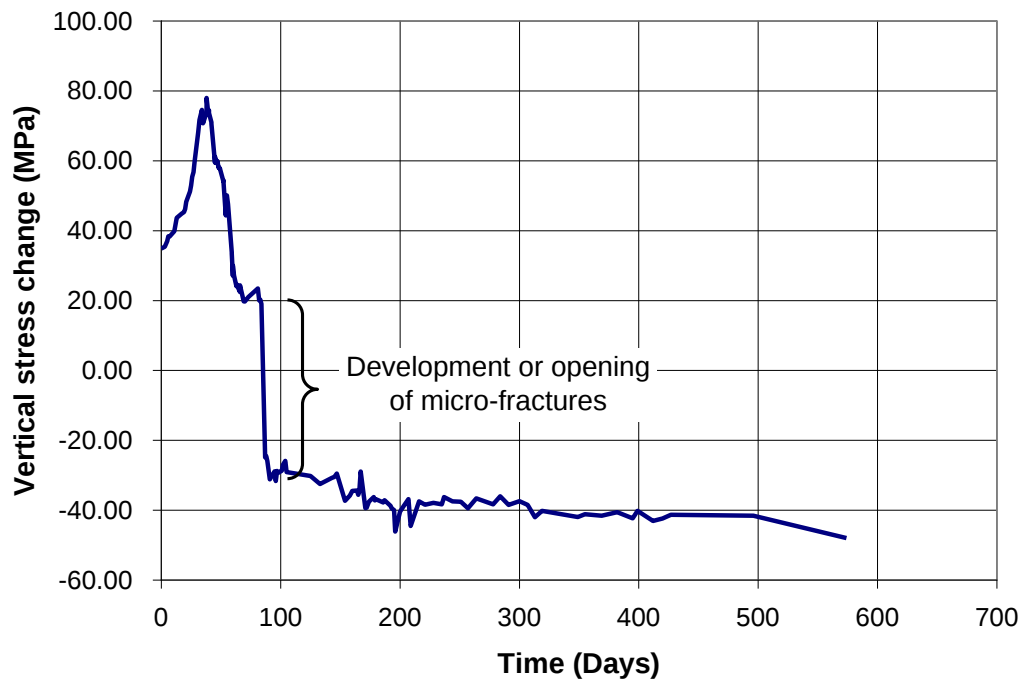


Figure 3-31 Impala site: absolute vertical stress change measurement approximately 4.5 m above Pillar 1 (Figure 3.2), showing an inferred stress drop at about 20 MPa. The additional strain is attributed to micro fracturing

3.7.6 Dependence on virgin stress levels

The results of a limited number of uniaxial tests performed on spotted anorthosite from different depths across Impala Platinum Mine (Figure 3-32) show a distinct change in the stress-strain behaviour at a depth of about 1000 m below surface. The severity of the nonlinear behaviour also appears to increase with depth below this point.

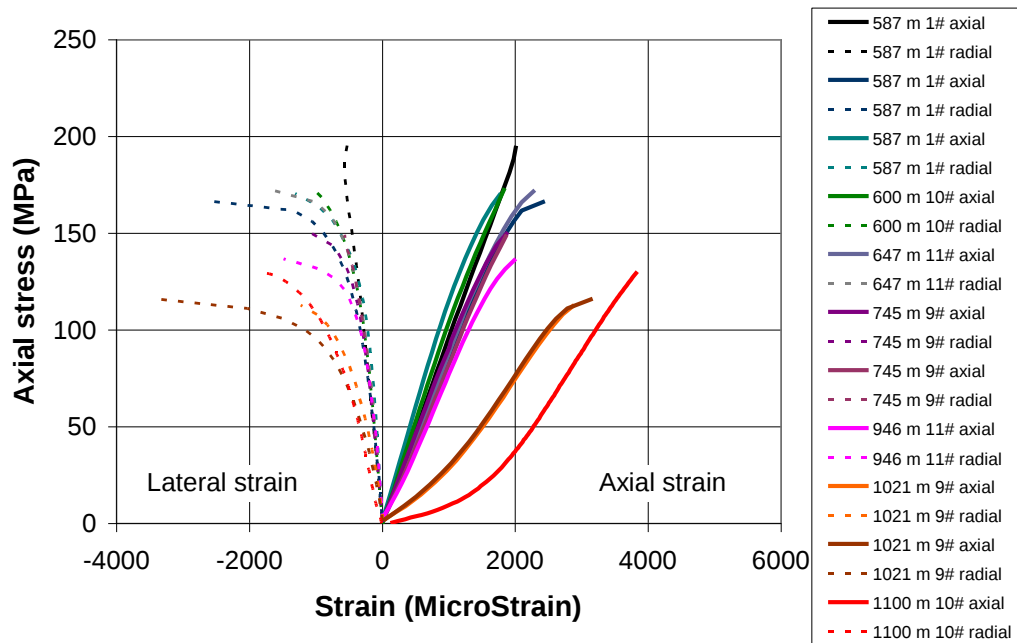


Figure 3-32 Uniaxial tests performed on spotted anorthosite from different depths below surface across the Impala Platinum Mine

The available evidence suggests that the nonlinear behaviour occurs as a result of closing and shearing of micro cracks. The underground measurements and the database of UCS tests suggest that these micro cracks open at a critical stress level, which can be related to a depth below surface at Impala Platinum. The mechanism appears to be opening of micro cracks or formation of micro fractures due to micro-scale stress differences that develop between crystals with different moduli when the virgin stress condition drops to some critical stress level. The theory assumes that the existing micro cracks from tectonic disturbances have healed and that the rock is stable in its virgin state. The critical change in the stress state of the rock mass due to mining is thought to cause deformation and micro-fracturing on grain boundaries. If the virgin stress conditions, therefore, are less than some critical level the strain differentials of adjacent grains are insufficient to cause micro-fracturing or opening of the healed micro cracks. Detailed microscope investigations are required to confirm this hypothesis.

3.7.7 Discussion

The results of the specialised laboratory tests and UCS database investigations suggest that:

- the stress conditions under which the samples were drilled affects the degree of nonlinearity;
- the large lateral dilation and low Young's modulus in the UCS tests can be partially explained by shearing on micro cracks;
- the rock mass becomes nonlinear if the stress drop due to mining is sufficiently large; and
- an *in situ* preferred orientation of micro cracks could not be established from the tests.

The tests suggest that the nonlinear behaviour occurs as a result of closing and shearing of micro cracks. Since the samples drilled at shallow depth do not show this nonlinear behaviour, there appears to be a critical stress drop threshold below which the nonlinear behaviour does not develop. The available evidence suggests that micro cracks open or micro fractures form due to micro-scale stress differences that develop between crystals with different moduli when the virgin stress condition drops by more than a critical amount. This fracture development is more likely in polycrystalline materials and could explain why, for example, this nonlinear behaviour is not usually observed on the more uniform quartzites in the hangingwall of the South African deep level gold mines (Figure 3-33). The theory assumes that the virgin rock conditions have stabilised after many years in the current stress state and that there are, therefore, minimal strain differentials between the grains before they are disturbed by mining or drilling.

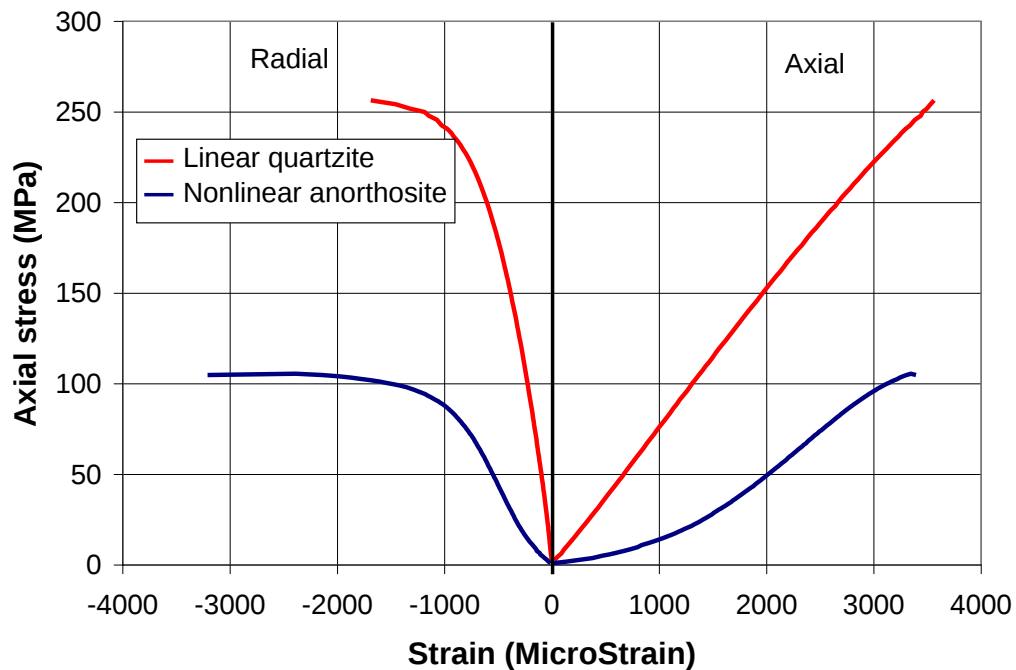


Figure 3-33 Typical UCS test result for quartzite from the South African deep level gold mines compared to a typical nonlinear anorthosite sample from the Impala site

Underground instrumentation at the Impala site suggests that micro cracks opened in the rock mass when the virgin stress condition dropped below about 10 MPa to 20 MPa. This means that zones in the hangingwall around the face and immediately above the pillars, where the stress had not dropped below this critical stress level, were linearly elastic with a relatively high modulus. However, areas where the vertical stress was less than about 10 MPa were nonlinear and with a much lower modulus.

Samples drilled from both the linear and nonlinear rock mass at the Impala site were nonlinear because the drilling process reduced the stress conditions in the core to below the critical level. Since the tested samples were always softer in the axial direction, it appears that the cracking process was enhanced by the field stresses. For this reason samples drilled in the linear elastic rock mass may have been comparatively softer due to the effects of the higher field stresses in those areas. It was, therefore, impossible to determine the true *in situ* conditions from UCS tests only, and it was necessary to measure strain change underground to establish this behaviour.

The stress condition at the time of sample extraction appears to be an important factor in determining the degree of nonlinear behaviour and is believed to induce the nonlinear strain relaxation process in rocks that would normally show a linear strain response to stress. Samples extracted from high stress conditions at the Amandelbult and Union sites generally showed nonlinear behaviour, whereas samples drilled under low stress conditions at these sites were linearly elastic. It appears, therefore, that the rock mass at the Amandelbult and Union sites is linearly elastic and the nonlinear behaviour was induced by the tensile stresses that developed at the drill tip during drilling.

The relatively small database of UCS tests from different depths across Impala Platinum Mine suggests a depth- or, probably, a virgin-stress dependence for micro-crack opening and nonlinear elastic rock behaviour. At Impala Platinum this depth appears to be about 1000 m. The opening of existing micro cracks or development of new micro fractures only appears to occur *in situ* under conditions where the virgin stress is sufficiently high. Further work is required to determine the stress condition or stress drop under which micro-fracturing initiates. Tests were performed to determine a preferred orientation of fracturing. However, the variation in the samples obscured any possible directional effects.

The effects of micro cracks on seismic-wave velocities and associated seismic evaluations need further investigations. Proper numerical simulations will need to account for the micro-fracturing and resultant softening of the rock mass around excavations. In addition, strain-gauge-based stress measurements need to be evaluated differently from the normal linear elastic materials. A methodology to evaluate measurements conducted in nonlinear material is suggested in Section 3.8.

3.8 Methodology to evaluate stress measurements conducted in nonlinear materials

The normal methods of determining stress from strain measurements in a rock mass assume that the rock mass is linearly elastic. The tangential modulus and Poisson's ratio at 50% of UCS are regarded as representative of the rock properties. However, these constants are not representative of a nonlinear material and, in particular, the Poisson's ratios are often greater than 0.7 at 50% of the UCS. The Young's modulus and Poisson's ratio vary significantly with stress and are also affected by confinement. The stress measurements, therefore, were evaluated with the use of laboratory tests conducted under similar loading conditions to the original measurements. These tests allowed for a direct correlation between the strains measured underground and a stress-strain curve determined in the laboratory. Biaxial tests were considered appropriate as the individual strain rosettes in both the *in situ* triaxial and biaxial (doorstopper) stress measurements were loaded biaxially underground as shown in Figure 3-34. Biaxial tests conducted on the original overcored triaxial samples were not used as the stress concentrations of a thick-wall cylinder are complicated by the nonlinear behaviour.

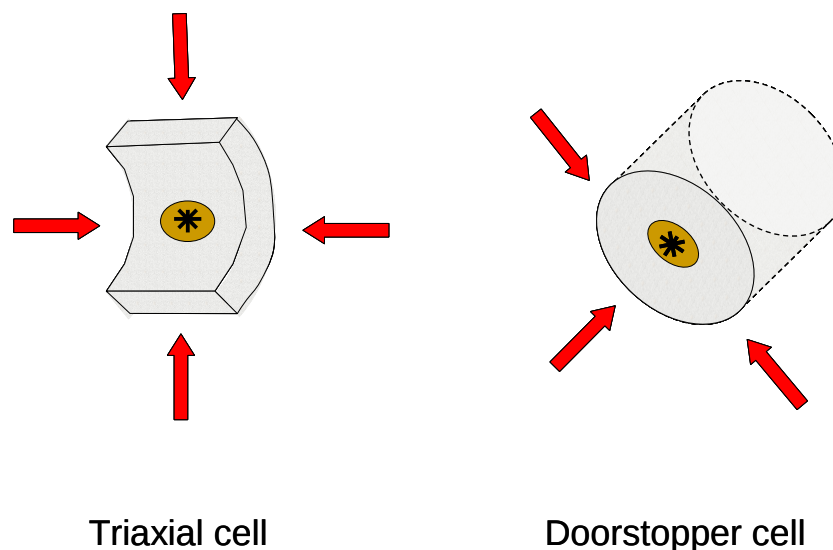


Figure 3-34 Diagram showing the *in situ*, biaxial loading conditions measured by the individual strain rosettes in the underground stress measurements. Triaxial left and biaxial right

The apparatus used in the biaxial tests is shown in Figure 3-35. A solid cylindrical-shaped sample was wrapped in a waterproof shrink sleeve and sealed around its ends with high-pressure seals. The cylinder and sample were pressurised with oil and the strains were monitored through the use of strain gauges cemented around the centre of the sample. Thus equal stresses were applied around the circumference and axial stress was zero. As many of the stress measurements, particularly in the vertical boreholes, showed similar strains in the radial directions, these tests were applicable. A separate modulus was determined for every stress measurement using Equation 3-2. The 'matrix' Poisson's ratio for the material was assumed to be the same in all the tests. Where the strains in the two principal directions were significantly different, the stiffnesses in these directions were also different, which complicated the stress calculations. The calculations in these instances were simplified by determining an average stiffness.



Figure 3-35 Apparatus used in the biaxial tests

$$E = \frac{(1 - \nu)\sigma}{\varepsilon}$$

3-2

Where:

E = Secant modulus

ν = 'matrix' Poisson's ratio

σ = Laboratory determined stress at the strain levels measured underground

ε = Strain measured underground

A normal laboratory UCS test was planned for every underground stress measurement. Regrettably there was a delay in receiving the results and most of the samples were tested before the implications of the nonlinear behaviour were realised. Therefore, only a few samples were available for biaxial tests. Fortunately, however, a reasonable strain-stress correlation was found to exist between the biaxial and UCS tests if the negative lateral and positive axial strains were added together in the UCS tests. The match of a 'modified UCS' test to a biaxial test of the same material is shown in Figure 3-36. The result suggests that the modified UCS tests are suitable for evaluating stress measurements in nonlinear materials.

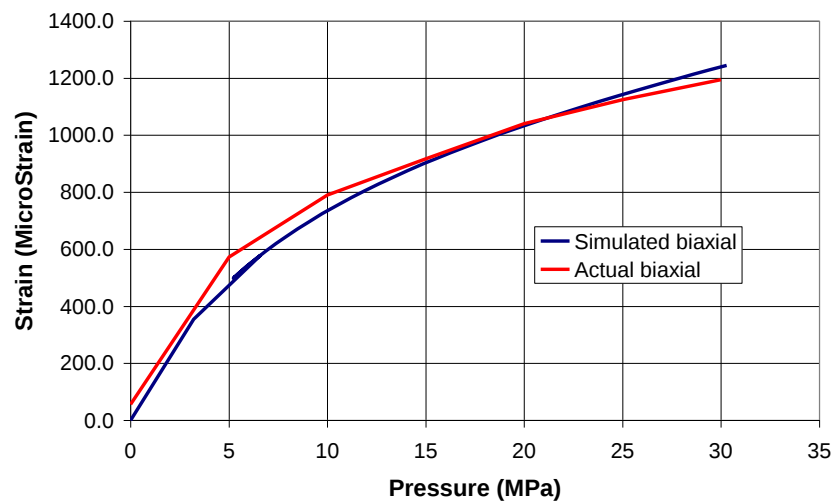


Figure 3-36 Comparison between the stress-strain relationship of a biaxial test and a simulated biaxial test, where the axial and radial strains of a uniaxial test were added together for each increment of stress. Both tests were conducted on anorthosite

The strain condition at the blind end of the borehole is affected by the stress acting along the axis of the borehole. Vreede (1991) developed corrections to extract the effects of the axial stress from the measurements (Equations 3-3 to 3-8). The effect of the nonlinear behaviour on these corrections was not investigated in this project, but is likely to be small. The 'matrix' Poisson's ratio was assumed appropriate and the corrections were made with the use of Equations 3-3 to 3-8. A detailed description of the evaluation process is provided in Appendix B. Axial stresses (σ_z) were estimated from triaxial stress measurements and numerical modelling.

$$\sigma_y = \left(\frac{a}{(b^2 - a^2)} \right) \left[\overline{\sigma_x} \left(\frac{b}{a} \right) - \overline{\sigma_y} + c \overline{\sigma_z} \left(1 - \frac{a}{b} \right) \right] \quad 3-3$$

$$\sigma_x = \frac{\overline{\sigma_x} - b \overline{\sigma_y} - c \overline{\sigma_z}}{a} \quad 3-4$$

$$\tau_{xy} = \frac{\overline{\tau_{xy}}}{(a - b)} \quad 3-5$$

Where: $\overline{\sigma_x}$, $\overline{\sigma_y}$ and $\overline{\tau_{xy}}$ are the measured stresses in the x and y directions.

The values for a, b and c were determined from FLAC (Itasca, 1993) modelling. Their relationship to the 'matrix' Poisson's ratio (ν) is shown in equations 3-6 to 3-8. The equations assume that the rock mass was elastic and that more work is required to determine the effects of the micro-fracturing on these values.

$$a = 1.33 + 0.1\nu \quad 3-6$$

$$b = -0.13 + \frac{\nu(1 - 1.5\nu + 5\nu^2)}{3} \quad 3-7$$

$$c = 0.37 - 1.1\nu \quad 3-8$$

3.9 Summary

A strong nonlinear stress-strain relationship was observed on all the tests from the Impala site, including cores retrieved from both high- and low-stress conditions. Similar behaviour was also observed in some samples from the Amandelbult and Union Section sites. However, at these sites the condition was associated with boreholes drilled into relatively highly stressed rock.

The nonlinear samples are elastic but the tangential modulus changes at every point along a stress-strain curve. These samples have a significantly lower

tangential modulus and a weaker strength than linearly elastic samples. In addition, nonlinear samples have very high Poisson's ratios, at 50% of their UCS.

High-magnification SEM investigations showed that the nonlinear elastic behaviour was associated with open micro cracks. Micro cracks were also observed in the linearly elastic rocks but no open cracks could be identified. These findings suggest that while micro cracks may be present in both linearly and nonlinearly elastic rocks, the nonlinear behaviour is associated with open cracks. However, only a limited microscope investigation was carried out and more work is suggested to confirm these findings. Tests were conducted to determine if a preferred orientation of cracks exists at the Impala site, but the variation within the samples did not allow a definite conclusion. However, the tests did show a definite influence of drilling on fracture development; i.e. the samples were always softer along their axes, which suggests more open fractures perpendicular to the core axis.

Nonlinear strain relaxation is normally associated with open micro cracks and appears to occur in Bushveld rocks under certain conditions. 2D elastic FLAC models were run to determine if cracks could explain the observed behaviour. A good correlation between the laboratory results and the models was observed, but the high stresses at which crack "closure" occurred in the laboratory tests could not be explained merely by crack closure theory. According to the crack models, the modulus of the unfractured material ('matrix' modulus) should have been measured once the virgin stress level was exceeded. However, the stress condition at which this modulus was measured appears to be much higher than the virgin stress conditions. The reason/s for this behaviour could be that the ratios of applied stresses were different from the original virgin conditions or that sliding on cracks contributed to the nonlinear behaviour. The sliding of cracks and associated opening of "wing-cracks" could also be an explanation for the high Poisson's ratios that were measured in the laboratory tests.

The high lateral strains and the hysteresis observed during cyclic loading also suggest shearing on micro cracks and opening of "wing cracks". It was discovered that both the 'matrix' modulus and Poisson's ratio can be identified at the start of an unload cycle. This phenomenon has been described previously

(Walsh, 1965). Kuijpers (1998) suggests that when the loading condition changes from static to unload, sliding on the cracks only initiates once sufficient strain has occurred to overcome the frictional effects on the micro-crack surface.

At Impala Platinum, nonlinear behaviour appears to be present at depths below 1000 m. This suggests a dependence on depth or a virgin-stress condition. In theory, micro cracks open or micro fractures form due to micro-scale stress differences that develop between crystals with different moduli when the virgin-stress condition drops by more than a critical amount. For this reason this fracture development is more likely in polycrystalline materials, such as the Bushveld rocks, than in more uniform materials.

Since the depth of the Impala site was 1100 m and all the samples were nonlinear, it appears that the stress conditions necessary for fracture formation/opening were present. Stress-change measurements conducted at the site suggested that the rock mass became nonlinear when the stress dropped below about 10 MPa. The nonlinear samples from the Amandelbult and Union sites were generally retrieved from high stress conditions. Conversely, linear elastic samples were drilled in areas of low field stresses. Thus micro fractures at these sites appear to have developed or opened as a result of the tensile stresses at the tip of the bit during drilling. The rock mass at the Amandelbult and Union sites, therefore, appears to be linear elastic.

The stress condition of the rock mass at the time of sample extraction appears to be an important factor in determining the degree of nonlinear behaviour and is believed to induce the nonlinear strain relaxation in rocks that would normally show a linear strain response to stress. More testing is required to investigate this issue.

The creep test result demonstrates the potential for time-dependent deformation. However, only one test was conducted for three hours and further work is required to determine the long-term effects of creep. The measured creep also does not prove that anelastic (time-dependent) strain recovery takes place during unloading. This issue needs to be investigated with the use of a combination of

acoustic monitoring techniques and strain-change measurements soon after the sample is drilled.

Nonlinear elastic rocks comprise an elastic 'matrix' with micro cracks. The true behaviour of a nonlinear elastic sample can be separated into 'matrix' elastic constants and an influence function based on a hyperbolic fit for compaction of a porous material. An equation was derived to characterise the behaviour of the cracks.

Cycle tests showed that the 'matrix' modulus and Poisson's ratio can also be estimated at the start of an unloading cycle of a stress-strain curve, if the pre-load on the sample is higher than the virgin stress state from which it was extracted. The reason for the absence of the nonlinear behaviour at the start of an unload cycle is not clear but can be explained by a lag in the reversal of shearing on micro cracks during the initial unloading. In other words, if the delay between the load and unload cycle were large enough, the normal nonlinear behaviour would be observed. Thus the 'matrix' constants could not be measured at the top of the unload curve of the creep test, since there was a time delay of three hours between the loading and unloading curves.

Stress measurements are particularly difficult to evaluate in nonlinear rock. It was therefore necessary to establish a methodology to interpret these measurements. This procedure was based on biaxial tests as these tests best describe the conditions under which the measurements took place. It was found that UCS tests can be used to simulate biaxial tests if the axial and radial strains are added for each increment of stress.

Stress-change measurements conducted in a linearly elastic rock mass were shown to respond to the strains of the rock mass even though a thin layer of rock around the borehole may have become nonlinear due to destressing. For this reason the evaluations of the nonlinear core adjacent to the stress measurements were not applicable. The stress-change measurements conducted over pillars at the Impala site were linearly related to strain until the stress dropped below about 10 MPa to 20 MPa.

The potential for strength reduction and fracture development needs to be comprehensively quantified, as it could negatively affect stability in the deeper mines. The effects of micro cracks on seismic-wave velocities and associated seismic evaluations and stress measurements also need further investigation. Proper numerical simulations will need to account for the micro fracturing and resultant softening of the rock mass around excavations.

In the next chapter, the nonlinear behaviour observed in the test laboratory is used to explain the rock mass behaviour at the Impala site. The results of the investigations described in Chapter 3 also enabled a meaningful evaluation of the stress measurements conducted under linear and nonlinear conditions at all three of the instrumentation sites. Chapter 4 describes the underground measurements and the interpreted rock mass behaviour around the open panels and pillars.

4 Rock mass behaviour around stopes

Chapter 3 discussed the nonlinear elastic behaviour that was observed on all laboratory samples from the Impala site and some specimens extracted from potentially high stress levels at the other two sites. The research suggests that the nonlinear condition had little effect on the behaviour of pillars as the behaviour only initiated when the stress level dropped below about 10 MPa at the Impala site. Therefore the immediate foundations behaved as a normal linear elastic material until failure was initiated. However, the interpretation of stress measurements was affected wherever the condition occurred and a method of interpreting these measurements was developed. The rock mass immediately surrounding the Impala panels was nonlinear and the influence of this condition had to be included in the analysis of the closure and deformation measurements in this chapter.

The research discussed in this chapter deals with the behaviour of the rock mass around pillars and open panels. Measurements of stress and deformation at three instrumentation sites were aimed at providing information on the interaction between pillars and pillar foundations. A specific literature review provides insights into the behaviour that could have been responsible for the deformation and stresses that were measured.

4.1 Rock mass behaviour: review of some relevant literature

Research conducted by York et al (1998) showed an interaction between the rock mass around the pillars and panels of a stope (foundations) and the pillars themselves. It was therefore necessary to study this interaction and the geomechanical behaviour of the foundations. Instrumentation was installed in and around selected crush pillars to determine both absolute and changes in stress and strain. This instrumentation also provided some insights into the rock mass behaviour around Merensky stopes.

A literature review on documented FOGs and findings from previous instrumented sites was conducted to provide a broad overview of the rock mass conditions around Merensky stopes. Since little instrumentation or monitoring of rock mass behaviour was done on the Merensky stopes in the past, little is known about the stope stability and failure mechanisms. Thus a survey of possible mechanisms of stability and failure were carried out. The relevance of these mechanisms was applied to the three instrumentation sites. Finally, a study of the literature dealing with the influence of pillars on foundations and how this influence would manifest in panels was carried out.

4.1.1 Reports on FOGs and observations

A total of 17 FOG reports were studied and detailed documentation of some of these events is included in Appendix C. Unfortunately, most of these reports covered incidents that occurred on the UG2 reef. Since the rock above the UG2 reef is stratified, it had an effect on the ground conditions, which is not usually the case above the Merensky Reef at shallow depth below surface. However, strata-parallel fracturing has often been observed above deeper level Merensky excavations. These fractures probably have a similar influence on the rock mass behaviour as the stratification does.

The primary and secondary mechanisms and release surfaces involved in the FOGs are summarised in Figure 4-1. Not one of the reported FOGs occurred as a result of a single factor. Some of the factors were vague but the investigation clearly shows that the majority of FOGs and collapses (80%) occurred on either shallow-dipping or curved (dome-shaped) discontinuities in conjunction with some other factor. A typical dome feature is illustrated by the FOG shown in Figure 4-2.

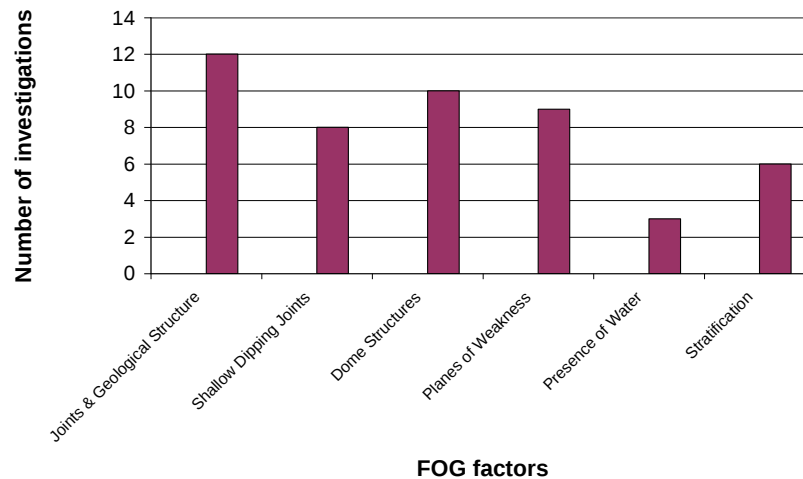


Figure 4-1 Primary and secondary FOG mechanisms



Figure 4-2 FOG from a dome structure

Surprisingly few cases reported the presence of water. However, water infiltration is often a secondary effect, indicating that movement has already occurred on a fissure at some height above the hangingwall.

The heights of all the investigated FOGs are shown plotted against the minor span of the excavation in Figure 4-3. The analysis showed a weak increase in FOG height with span but surprisingly high FOGs were recorded in the spans between 10 m and 20 m.

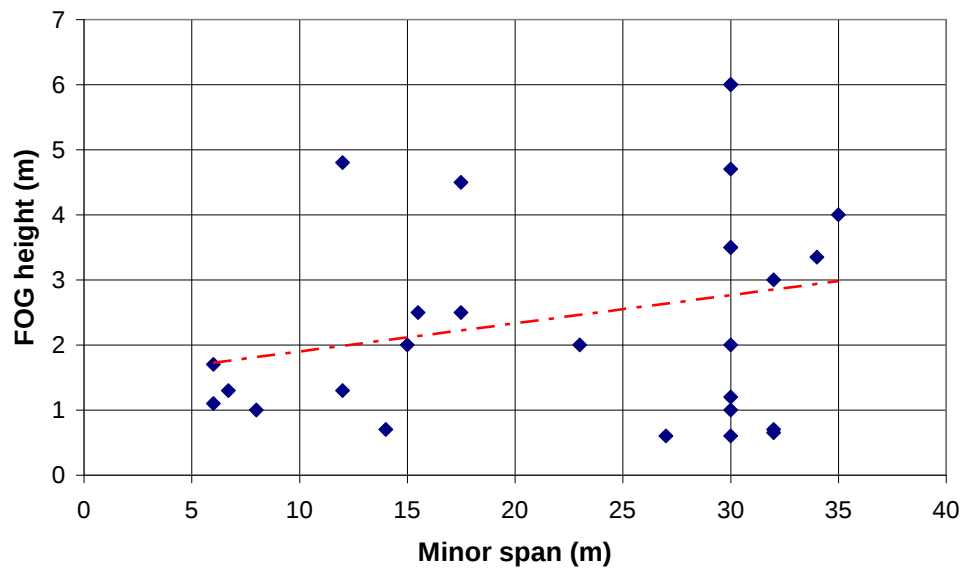


Figure 4-3 FOG height related to minor panel span

Watson (2000a) and Watson (2000b) report stress fracturing in a Merensky Reef hangingwall adjacent to a pillar at shallow depth (300 m to 600 m below surface). The fracturing described in Watson (2000b) appears to have arched from the pillar (Figure 4-4) up into the hangingwall of the panel and resulted in unstable conditions. A similar stress-induced fracture was described in an instrumentation site at Impala Platinum at a depth of 1100 m below surface (Watson et al, 2009). Evidence of high horizontal stress at shallow depth was also shown by Watson (2000c), where gothic arching and associated fracturing were observed in an in-panel raise prior to stoping (Figure 4-5). In one instance, a large FOG occurred from a stress-induced fracture that developed above a face (Watson, 2000d). This was evidence of high horizontal stress in the direction of mining. The fracture appears to have followed the major stress trajectory from the vertical solid face, curving over towards the stope (Figure 4-6). It appears from the fracture surface that once the initial fracture had developed above the face, the resultant cantilever was assisted by gravity and the fracture was extended horizontally over the stope under tension.



Figure 4-4 Horizontal stress fracturing observed in the hangingwall of a panel, adjacent to a pillar



Figure 4-5 Gothic arching and extension fracturing observed in the hangingwall of an in-panel raise

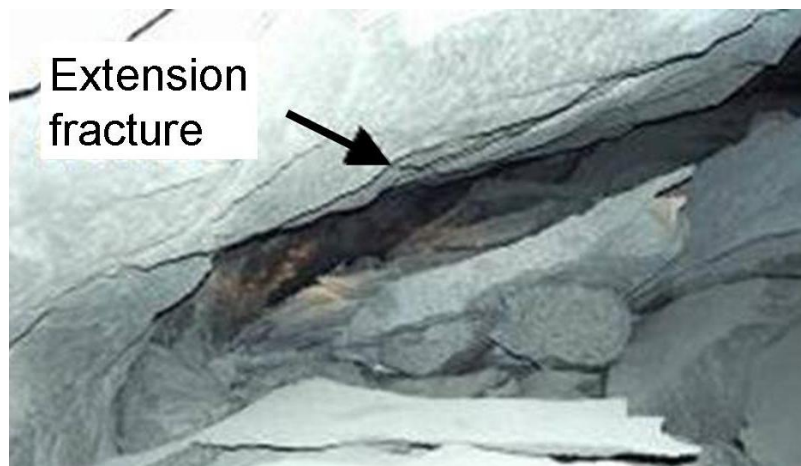


Figure 4-6 FOG from an extension fracture that developed above the face

Extensometer measurements at Northam Platinum Mine (Roberts et al, 1997) showed inelastic deformations up to 25 m above a stope with spans of 70 m x 120 m at a depth below surface of 1300 m. Open discontinuities were observed to a height of 30 m above a 120 m x 160 m stope at Union Section in a borehole camera survey (Du Toit, 2007). This stope was also about 1300 m below surface. Both these stopes were mined without in-stope pillars or backfill and provided similar symptoms of a pending backbreak as observed in the shallow mines (Roberts et al, 1997). Roberts et al (2005) showed that a support resistance of about 1 MPa was sufficient to prevent such a backbreak. This support resistance is generally supplied by the residual strength of 'crush' pillars or backfill (Roberts et al, 1997). Watson (2003) observed open discontinuities up to a height of 4.4 m above a 35 m-wide panel between abutments at Union section (depth below surface was 1200 m). The borehole was drilled prior to mining and the fracture positions could, therefore, be checked against the borehole core. In particular, the discontinuity observed at 4.4 m was not in the original core, but was near a change in lithology. It appears, therefore, that strata-parallel fractures developed in the hangingwall during mining. These fractures probably developed as a result of differential strains between different rock types during relaxation of the strata. Similar fracturing was reported by Swart et al (2000). In addition, apparent "stratification" above the Merensky Reef may occur as a result of igneous intrusion/extrusion or metamorphic flow processes. Such weaknesses were apparent in tensile tests performed on hangingwall pyroxenite from a Merensky stope at Union Section (Watson, 1996). More recently, parting was observed at

4.5 m above a 30 m-wide panel at Union Section (Du Toit, 2007). Open discontinuities were observed in borehole camera surveys up to about 20 m above 30 m wide panels on the Merensky Reef at Impala Platinum (Fernandes, 2007).

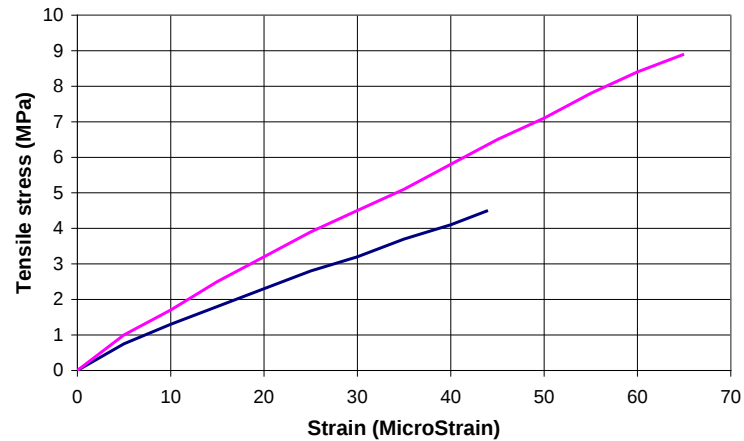


Figure 4-7 Direct tensile tests performed on pyroxenite from the hangingwall of a stope at Union Section. The weaker sample probably failed on a weak plane, which was not visible

Stope collapses that appeared to be associated with high horizontal stress were often also linked to potholes and anorthositic rocks in the immediate hangingwall. These potholes are usually oval in shape and are associated with a drop in the elevation of the reef in the area in the pothole. For this reason the shallow-dipping tabular stopes are often mined into hangingwall strata in the area of the pothole. Stress measurements conducted in a pothole at Lebowa Platinum Mine (Watson et al, 2005b) showed very high horizontal stress (100 MPa) at shallow depth (250 m below surface).

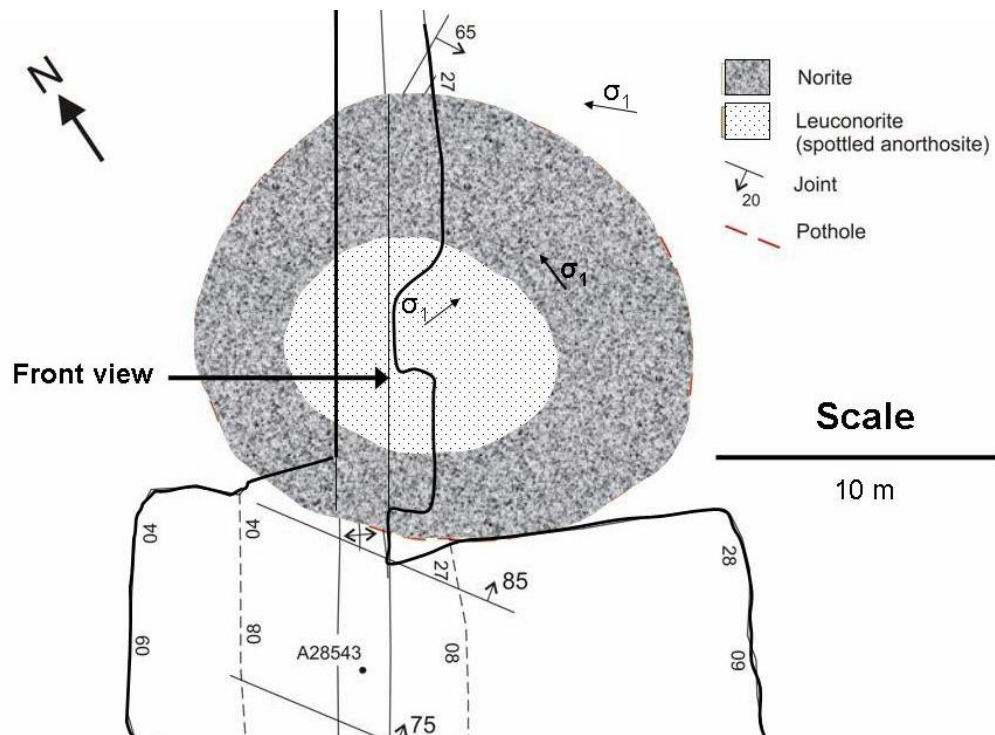


Figure 4-8 Plan view of a pothole at Lebowa Platinum Mine

It should be mentioned that most Merensky stopes are stable with a support system of 200 mm diameter mine poles spaced approximately 2 m x 2 m and crush pillar lines spaced about 30 m apart. Extensometers installed at a shallow-depth site (400 m below surface) indicated that the hangingwall behaved in a pure elastic manner (Watson, 1998). At a deeper level sites, however, strata parallel fractures were monitored, e.g. Impala at 1100 m below surface (Watson et al, 2009) and Union Section at 1200 m below surface (Watson, 2003).

Since little was known about the rock mass behaviour at most of the FOG and observation sites, failure mechanisms were hypothesised in the reports. A literature review of the possible mechanisms of stability and failure is set out in Section 4.1.2 and its findings were used to help with the interpretation of the instrumentation measurements conducted at the three sites described in this thesis. The on-going occurrence of FOG incidents on the Bushveld mines highlights the need for a better understanding of rock mass behaviour.

4.1.2 Stability and instability mechanisms

Under ideal conditions, the rock mass behaves as an elastic medium, requiring only the stabilisation of keyblocks or wedges. The formation of beams is common in Bushveld Complex mines (Swart et al, 2000). Such beams were observed in deeper level excavations at Union Section, Richard Shaft (Watson, 1996) and Impala Platinum 1-shaft (Watson, 2003). The majority of FOGs on the Bushveld, however, occur from unsupported or insufficiently supported wedges and blocks. Most of these wedges and blocks form from shallow-dipping and curved discontinuities as described in the previous section.

4.1.2.1 Vertical tensile zone

In several instances the FOG reports suggested that parting occurred above the vertical tensile zone (VTZ), which is theoretically impossible. All these partings occurred on shallow-dipping or curved structures, which indicates that the height of the VTZ does not restrict the fallout height if these structures are present. A 2D ELFEN model was set up to consider the effects of the VTZ on an isolated panel in the presence of vertical and horizontal discontinuities. The model and its results are described in Watson et al (2007c) and shown in Appendix D. The findings suggest that when the pillars between panels crush there is a significant increase in the height of the VTZ, commensurate with a decrease in the load-bearing capacity of the pillars (as expected). The VTZ height then becomes dependent on the residual strength of the pillars rather than depth below surface. Furthermore, it should be noted that the VTZ is independent of the horizontal stress. The VTZ can extend up to about 5 m above a stope where individual panels of 30 m are separated by ‘crush’ pillars. The modelling suggests that if parting occurs within this height, the effective height of the VTZ is increased.

4.1.2.2 Effects of a persistent shallow-dipping discontinuity

A 2D UDEC model (with the Barton–Bandis joint addition) was used to determine the behaviour of a rock mass influenced by a shallow-dipping, persistent discontinuity (Figure 4-9). The model was calibrated by a UG2 instrumentation site and its results are described in Watson et al (2007c). The model comprised

weak chromitite parting planes at 1.6 m, 2.3 m and 3 m (triplets) and very weak, calcite-filled planes at 5 m and 7 m. These two horizontal planes and the discontinuity cutting through the strata are shown as UCB planes in Figure 4-9.

The findings of the research showed that with normal mine pole support, a collapse occurred up to 5 m above the stope at a span of 13 m. However, if support with sufficiently high pre-stress load was installed (100 kN/m^2), the hangingwall was stabilised. Provided this support was installed within 2.8 m of the face, spans of greater than 35 m could be safely mined, even though the support resistance was insufficient to support the deadweight height of 7 m.

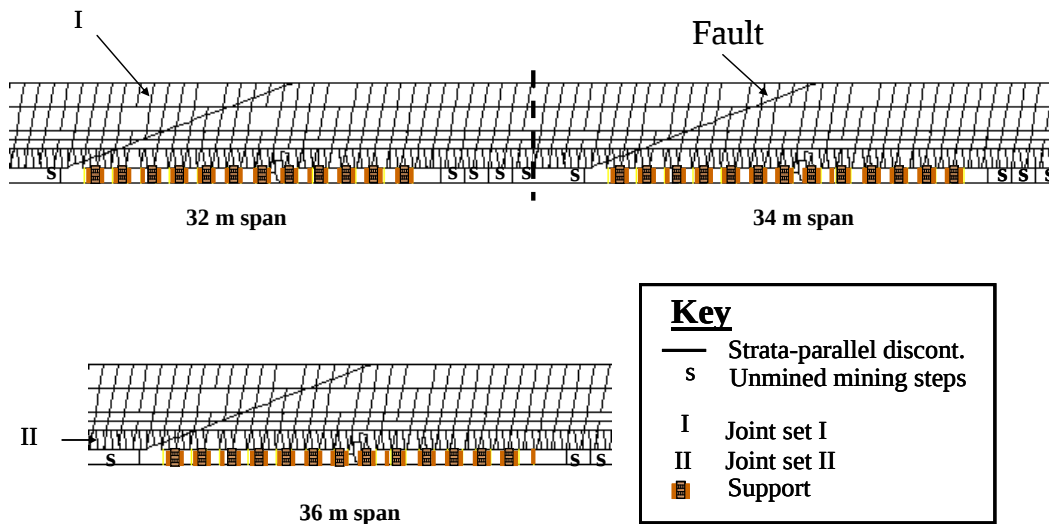


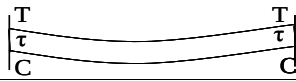
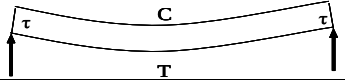
Figure 4-9 Discontinuity configuration and results of the UDEC model when a support resistance of 100 kN/m^2 was provided at installation. The support-to-face distance was kept to a maximum of 2.8 m

4.1.2.3 Beam theory

Elastic beam theories assume that the rock above an excavation acts as a series of beams or plates loaded by self-weight. The roof span is designed so that an allowable stress is not exceeded in these beams or plates (Swart et al, 2000). Insights into the maximum horizontal stresses that may develop are provided by the equations in Table 4-1. These solutions apply to thin beams – i.e. $t < L/5$. The ratio of tensile to shear stress in thin beams is greater than 3 and, therefore,

tensile failures dominate. Axial stresses (also called the fibre stress) at the centre of the beam are tensile and compressive at the bottom and top respectively. The locations of the maximum compressive, tensile and shear stresses are shown in Table 4-1.

Table 4-1 Beam solutions for thin beams (after Ryder and Jager, 2002)

	Built-in both ends	Simply-supported
		
Max. deflection ($\delta_{\max}$)	$\delta_{\max} = \frac{\rho g L^4}{32 E t^2}$	$\delta_{\max} = \frac{5 \rho g L^4}{32 E t^2}$
Max. fibre stress (CT)	$CT = \frac{\rho g L^2}{2t} + \sigma_h$	$CT = \frac{3 \rho g L^2}{4t} + \sigma_h$
Max. shear stress (τ)	$\tau = \frac{\rho g L}{2}$	$\tau = \frac{\rho g L}{2}$
σ_h for buckling	$\sigma_h = \frac{\pi^2 E t^2}{3 L^2}$	$\sigma_h = \frac{\pi^2 E t^2}{12 L^2}$

Where:

ρ = density, g = gravitational acceleration (9.81 m/s²), L = span (m), t = beam thickness (m), σ_h = virgin horizontal stress, $\delta_{\max}$ = the maximum deflection, E = Young's Modulus, CT = maximum tensile and compressive stresses at the top and bottom of a beam, τ = maximum shear stress, σ_h = horizontal buckling stress

The horizontal stress at the centre span of a beam with built-in ends is half the maximum stress at the edge (Diederichs and Kaiser, 1998). In the absence of appropriately oriented natural joints, Diederichs and Kaiser (1998) suggest that vertical fractures may develop in a beam during mining, due to the low tensile strength of most rocks (0,05 to 0,1 x UCS). An unjointed beam with built-in ends, loaded under its own weight, therefore, will yield when the maximum tensile stress above the edge of the beam (over the abutments) exceeds the tensile strength of the rock. Vertical tensile fractures form at the abutments and the

beam progresses to a simply supported beam (assuming the beam is properly supported above the abutments). Under conditions where the hangingwall beam contains vertical fractures or natural vertical joints in a low horizontal stress environment, the use of the elastic beam formula is invalid (Ryder and Jager, 2002). The structure can, however, be analysed as a Voussoir beam when the angle between the plane of the cross-cutting joints and the normal to the parting plane is less than one third to one half of the effective friction angle of these joints (Diederichs and Kaiser, 1998).

The maximum tensile stress in a freely supported beam is at the bottom centre of the beam. Prior to beam failure, fracturing will occur at the mid-span (bottom of the beam) and the beam becomes a Voussoir beam. The transition from a continuous elastic beam to Voussoir beam is normally assured if the tensile stresses exceed the tensile strength of the rock (Diederichs and Kaiser, 1998). A stable set of key-blocks locked into a compressive arch is known as a Voussoir beam (Figure 4-10).

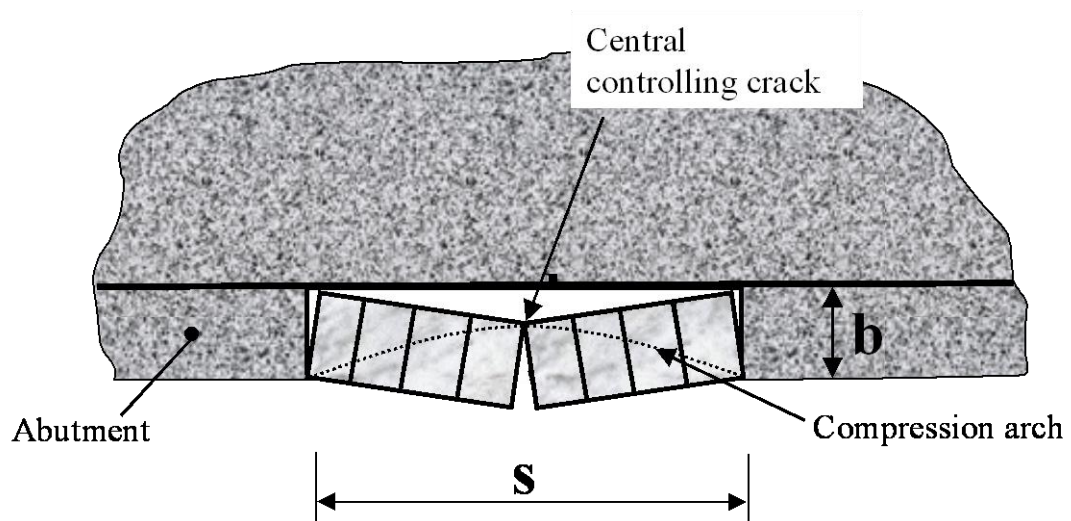


Figure 4-10 Geometry of a self-standing Voussoir beam (after Daehnke et al, 1998)

A Voussoir beam develops when joints or fractures open at the abutments and mid-span during deflection (Ryder and Jager, 2002). The central transverse crack in a discontinuous hangingwall beam determines the deformational behaviour (Ryder and Jager, 2002). Additional deadweight may load the top of the beam

and this can be accounted for in the same way as in the elastic beam (by adjusting the density, ρ).

Some of the stress measurements that were carried out at the instrumentation sites suggested beam formation in the hangingwall. However, the additional contribution of the beams to the elastic convergence that was measured in the stope was negligible. A much larger inelastic component of closure was generally associated with extension fracturing and possible buckling of the footwall. Voussoir beams may have developed in the hangingwall pyroxenites at the Union site but not at the other two sites.

The shear stress, s , on a transverse cross-section through an elastic beam is always a maximum at the supported end, as shown by the built-in end and freely supported beams in Table 4-1. Thus, relative horizontal movement along the interfaces of different hangingwall strata is likely to occur close to the ends of the beam (Ryder and Jager, 2002). A potential influence of these shear stresses on the horizontal stress measurements is discussed in Section 4.3.

4.1.2.4 Buckling theory

Buckling is possible in high-stress environments, where long narrow slabs have developed (Ryder and Jager, 2002). Fairhurst and Cook (1966) suggests that brittle-buckling failure occurs when a rock is stratified in such a way as to separate portions close to a free face into long, continuous and narrow slabs, which will buckle due to an axial load (Figure 4-11). The process of brittle-buckling failure could be approximated as follows (Jeremic, 1987):

- under the action of the major principal stresses a rock slab will split along the planes of stratification and will form a rock “beam”, where its thickness is small compared to its length; and
- the slab will buckle as a result of eccentric axial loading in compression.

Buckling could conceivably occur along extension fractures formed in the foundation of a tabular stope. Such fracturing was observed in the footwalls of all three of the instrumentation sites described in this thesis.

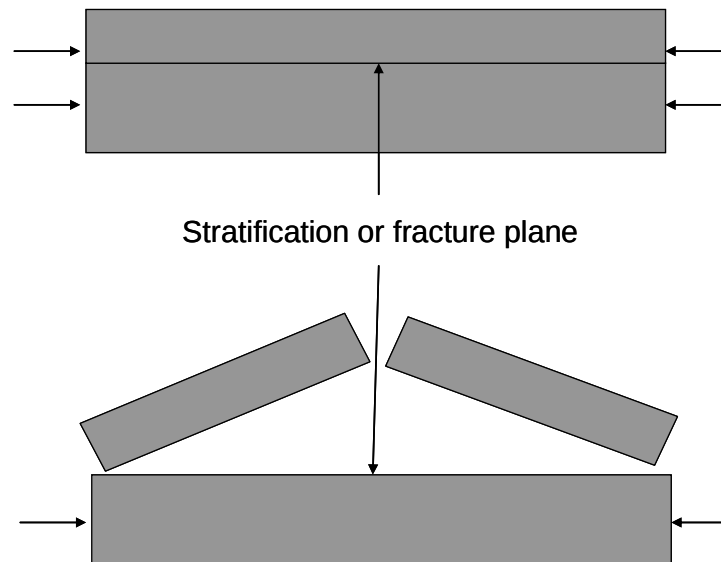


Figure 4-11 Buckling due to eccentric loading

4.1.2.5 Extension fracture theory

In its broadest terms, the hypothesis of the 'Extension Strain Criterion' is that the lateral 'extensional' strain of a material at failure is limited to a (material or even universal) constant value – i.e. final rupture takes place when a critical dilatatory strain (ϵ_c) of a material is exceeded (Ryder and Jager, 2002). For example, a quartzite having a Young's modulus of 70 GPa, a Poisson's ratio of 0.2, and an in-situ uniaxial strength of 70 MPa will sustain a lateral strain of 0.2 mm/m at failure under uniaxial loading, and this could be taken to be the approximate value of the critical extension strain (ϵ_c). Extensive testing performed as part of the anorthosite project (Watson et al, 2005b) suggests that the extension strain criteria limits for anorthosites are between 0.16 mm/m and 0.2 mm/m. The fractures will form in planes normal to the direction of the extension strain, which corresponds to the direction of the minimum principal stress. The critical strain in this direction is described by the following formula (Stacey, 1981):

$$\varepsilon_c = \frac{1}{E} [\sigma_{3c} - \nu(\sigma_{1c} + \sigma_{2c})] \quad 4-1$$

and extension fracturing will occur if:

$$\nu(\sigma_{1c} + \sigma_{2c}) > \sigma_{3c} \quad 4-2$$

Where:

σ_{1c} is the major stress

σ_{2c} is the intermediate stress

σ_{3c} is the minor stress

In practice, fractures initiate at 25 to 30% of the ultimate rock strength (Stacey, 1981). Unfortunately, the stress condition in the footwalls of the instrumentation sites was not determined. However, evidence from sockets (Watson, 2005a) and gothic arching suggests that the horizontal stresses that could have caused the observed fracturing were probably within the critical 25 to 30% range. This theory, therefore, seems to provide the most plausible explanation for the observed footwall fracturing.

The applicability of the 'Extension Strain Criterion' to observed fracturing and rock failure underground has been suggested several times in rock engineering literature (Wagner, 1980; Stacey, 1982). The theory does not, however, describe the mechanism of failure, but only provides a prediction under which failure is likely.

The theories of stability and instability do not provide insights into the influences of pillars on the foundations. Some numerical modelling and laboratory experiments are described in the literature and a summary is provided in Section 4.1.3.

4.1.3 Influence of pillars on foundations

2D FLAC modelling (Itasca, 1993) indicated four modes of foundation failure (York et al, 1998). The models were calibrated on the results of punch tests performed by Wagner and Schümann (1971) and Cook et al (1984). Weak strata-parallel parting planes were included in some of the models and these planes were shown to be a major contributing factor to the failure modes shown in Figure 4-12. Excessive compressive load on the pillar resulted in foundation fracturing, including wedge formation, which caused horizontal dilation and possible buckling of the stope foundation. York et al (1998) suggested that the mechanism of foundation failure depends on the stiffness of the system. The stiffness of the system is influenced by factors such as the span-to-pillar width ratio (s/w) and the depth of parting (h). The models assumed half symmetry conditions – i.e. repeating geometries of stopes and pillars with infinitely strong, 3 m-wide pillars and 30 m-long panels. Descriptions of the failure mechanisms shown in Figure 4-12 follow the figure.

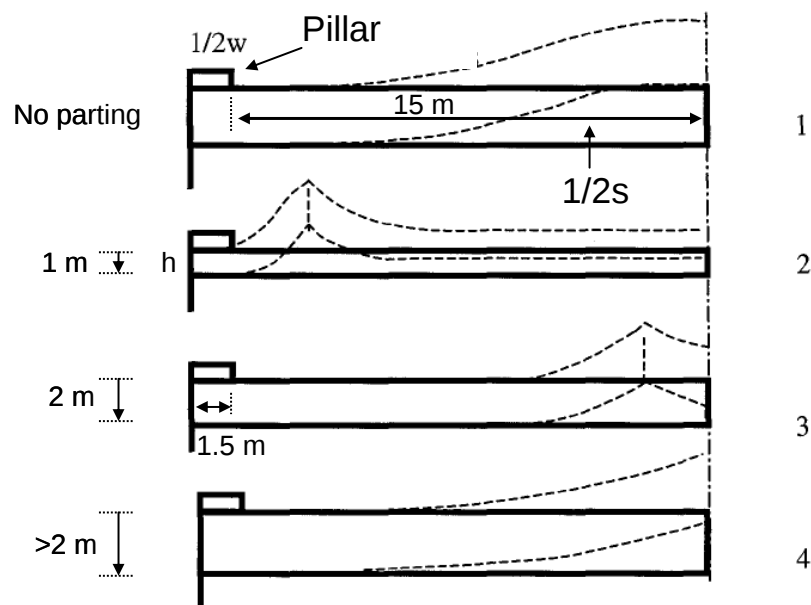


Figure 4-12 Foundation failure mechanisms (York et al, 1998). Quarter symmetry applied in the model. Solid lines show the model geometry; dashed lines indicate the buckling

Mechanism 1: the buckling/deformation has two distinct curvatures and the maximum deformation is reached at the mid-span. The stiffness of the system results in high rigidity in the vicinity of the pillar.

Mechanism 2: the buckling results in maximum deformation in the vicinity of the pillar. In this case the rigidity of the pillar foundation system in the vicinity of the pillar is low. The mechanism has been mostly observed for shallow partings such as 1 m into the footwall.

Mechanism 3: the buckled parting has its maximum deformation in the middle third of the span. The rigidity of the pillar foundation system in the vicinity of the pillar is higher than in Mechanism 2. This is the case where the s/w and h parameters are both in the middle of their ranges. This type of failure was observed for cases where the parting was 2 m below the footwall.

Mechanism 4: similar to the first mechanism in terms of stiffness of the system, this mode of failure is characterised by the curvature shown by “4” in Figure 4-12. The maximum deformation is much larger than in Mechanism 1, and is also reached at mid span. Thicker partings are expected to behave this way, especially at depths below surface of greater than 1000 m.

York et al (1998) found that the s/w and h are the most influential parameters in determining the mode of failure, but other parameters such as k -ratio, depth and friction angle also play a role. The mechanisms were based solely on 2D models but show that the whole panel can be affected by pillar punching and foundation failure. Inelastic closure often occurs in Merensky panels and may be the result of pillar punching or fracturing in the footwall. Under certain conditions, these pillars have been known to affect the hangingwall as well (Van Aswegen, 2008).

The term “buckling” used by York et al (1998) to describe four behaviour modes of foundation failure may, in some cases, be inaccurate. Mechanism 1 appears to describe elastic convergence or elastic-beam bending and is therefore not a failure mechanism but rather a deformation mode. Mechanism 2 may be a type of buckling but could also be related to the formation of a Prandtl's wedge (Prandtl, 1921). Mechanisms 3 and 4 are unlikely to have buckled because of the

thickness of the beams. A more likely explanation for the observed behaviour is a bending beam that developed a tension crack.

As part of the research conducted for this thesis, FLAC models were run to determine realistic pillar behaviour with the influence of the surrounding foundations. The models were calibrated from the underground measurements at the Amandelbult site. In the following section, the modelled interactions between the pillars and the rock mass surrounding the stope are described in detail, and the pillar behaviour is described in Chapter 5.

4.2 Modelling of foundation damage around a Merensky pillar

The literature review discussed in Chapter 1 shows inter-dependence between the behaviour of the rock mass around a stope and the pillars within the stope. A series of FLAC (Itasca, 1993) models were constructed to investigate the influence of the pillar foundations on pillar behaviour. For the purposes of the model, the foundation material properties were assumed to be the same as the pillar, which was a reasonable approximation for the pillars at all three sites. The model included the pillar itself and the immediate hanging- and footwalls. This section concentrates on the potential foundation damage and the results of the modelled pillar behaviour are discussed in more detail in Chapter 5.

One important parameter that needed to be considered for pillar behaviour was material brittleness. In the models, brittleness is defined as the rate of stress decrease after failure. In these models, post failure behaviour is controlled by cohesion loss. Therefore, a direct relation between cohesion softening (strain softening) and material brittleness can be quantified. The internal friction angle and the dilation angle were not varied in these models to avoid additional complications.

In the numerical models brittleness is unfortunately also affected by grid size, since it influences failure localisation. Therefore the model brittleness is controlled by both the post failure strain softening and the grid size. This issue was considered in the model analyses. While brittleness does not affect the strength of typical slender uniaxial and triaxial test specimens, it becomes very relevant in (pillar) geometries with larger w/h ratios. In the larger w/h ratio geometries, failure progresses from the edge towards the core of the pillar even before the peak pillar strength is reached. In the case of a comparatively brittle material, the failed material will rapidly lose its strength and failure can progress relatively easily into the pillar. Failure initiation and peak strength will be of similar magnitude. However, a more ductile material will maintain some strength after failure and failure progression into the pillar will be retarded. The core of the pillar will be confined by the partially failed material and is able to sustain a higher load.

As a consequence the magnitude of failure initiation will be much lower than the peak strength.

Table 4-2 shows the Mohr-Coulomb parameters that were conventionally calibrated from laboratory tests (Ryder and Jager, 2002). However, the brittleness parameters had to be adjusted to match observed pillar behaviour, including strength. Two sets of models were run: those represented by a more ductile material (S) and calibrated by underground measurements of Pillar 1 at the Amandelbult site (Chapter 5.6); and those represented by more brittle material (B). The value ε_{pr} refers to the plastic strain at which the residual strength is reached.

Table 4-2 Material and FLAC model properties (calibrated from underground measurements)

	σ_3 (MPa)	Co (MPa)	ϕ_0	ϕ_{res}	ε_{pr} (10^{-3})	ψ_0	ψ_{res}
A	2	15	55	50	< 0	-	-
	5	15	55	50	< 0	-	-
	10	15	55	40	< 0	-	-
P	2	16	52	50	0.75	-	-
	10	16	52	40	0.65	-	-
B		20	40	40	25	10	10
S		20	40	40	100	10	10

A = anorthosite lab. Test,

P = pyroxenite lab. Test,

B = brittle model,

S = ductile model,

Co = cohesion,

ϕ_0 = internal friction angle at peak load,

ϕ_{res} = residual internal friction angle,

ε_{pr} = residual plastic shear strain,

ψ_0 = Dilation angle at peak load and

ψ_{res} = residual dilation angle.

The reasons for the differences between the test and model parameters in Table 4-2 are as follows:

- In the laboratory tests the calibration of strain softening parameters is based on a uniformly stressed and strained specimen. This is not realistic as failure localisation will cause a concentration of straining. Therefore, the parameters do not represent the actual behaviour in the failure zones.
- In the models the brittleness was also influenced by the grid size, which needed to be considered in the quantification of the relevant parameters.
- A constant internal friction angle was selected to keep the model simple.

Brittleness is simply represented in the model by a linearly decreasing cohesion value in relation to induced plastic strain (Figure 4-13). This simple model essentially allowed for a sensitivity analysis on brittleness.

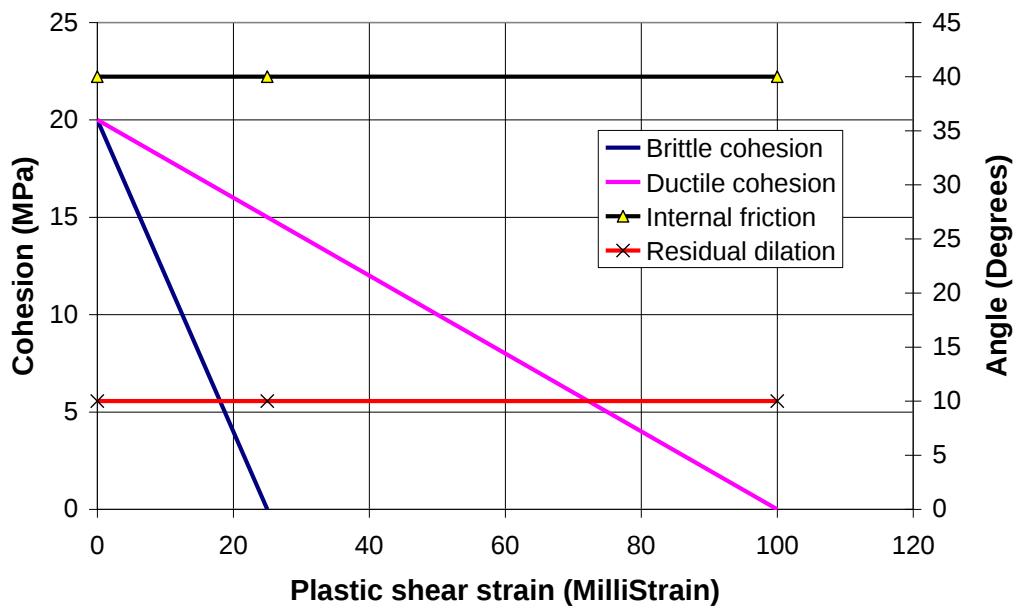


Figure 4-13 FLAC model properties (the more ductile model was calibrated from underground measurements)

Boundary conditions play an important role in the punching mechanism, as they affect horizontal confinement. In the models (Figure 4-14), the vertical boundaries are not allowed to move in a horizontal direction (thus simulating a fully replicated set of pillars). The presence of discontinuities such as bedding planes, faults and joints should also affect the punch resistance, but this was not investigated in the

models. While the numerical models provide insight into the failure mechanisms, it must be emphasized that these models always need to be calibrated against realistic data. Mesh density and rate of softening are important parameters in this respect and they cannot be arbitrarily selected. Table 4-2 shows the parameters that were used in the numerical models, as well as the parameters that are obtained from triaxial compression tests on hangingwall pyroxenite and footwall anorthosite from the Impala site (Figure 4-15 and Figure 4-16). The plastic strain at which the residual strength is reached (ε_{pr}) was much larger in the numerical models than determined by the laboratory tests. This finding suggests more ductile behaviour of the model material. The reason for the choice of the model parameters is that the failure localisation in the small cylindrical-shaped samples that were used in the laboratory tests is different from that of the pillar models. In the laboratory samples the strain softening and failure only takes place on (an) individual fracture(s). However, the actual strain-softening rate along these fractures is not quantified/measured, as the monitored stress-strain curve represents the average behaviour of the fracture(s) together with the elastic “undamaged” rock. The actual strain-softening rate along the fracture is therefore much larger. It was not possible, therefore, to obtain realistic post-failure parameters from the laboratory tests. It should be emphasised that the models cannot be expected to reproduce realistic failure localisation. Even if realistic post-failure parameters were to be used in the model, the model results would still not be realistic as the fracture geometry will be different from the complex fracture pattern in a full-scale *in situ* pillar. At best, the laboratory results can be used to calibrate parameters for models of these laboratory tests, but they cannot be expected to be representative of pillar failure. Therefore, the selection of the strain-softening parameters is somewhat arbitrary and can only be calibrated against *in situ* pillars or appropriate laboratory pillar models. In addition, it should be appreciated that the element size (mesh density) affects the strain-softening rate in the model. Larger mesh density is associated with a more brittle behaviour when failure localisation occurs.

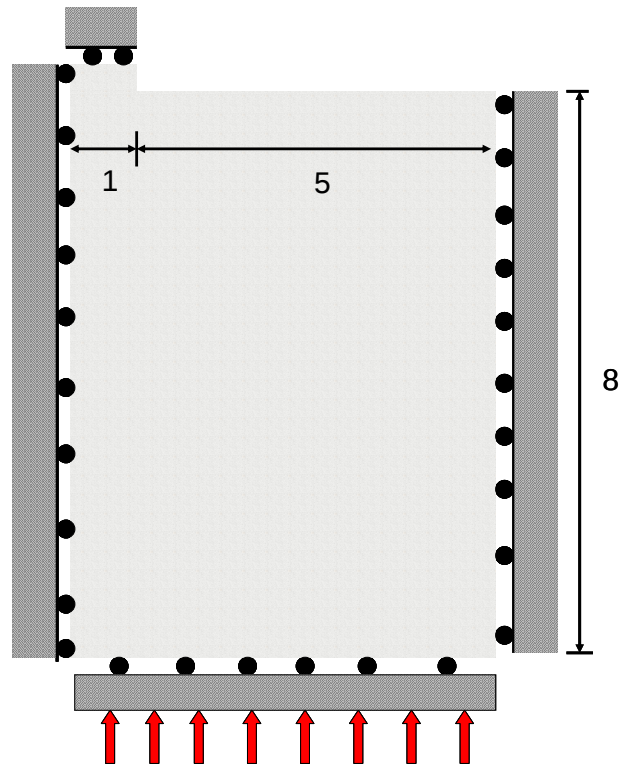


Figure 4-14 Diagram showing the double symmetry FLAC model used in the pillar and foundation investigations. The model was loaded along the bottom edge

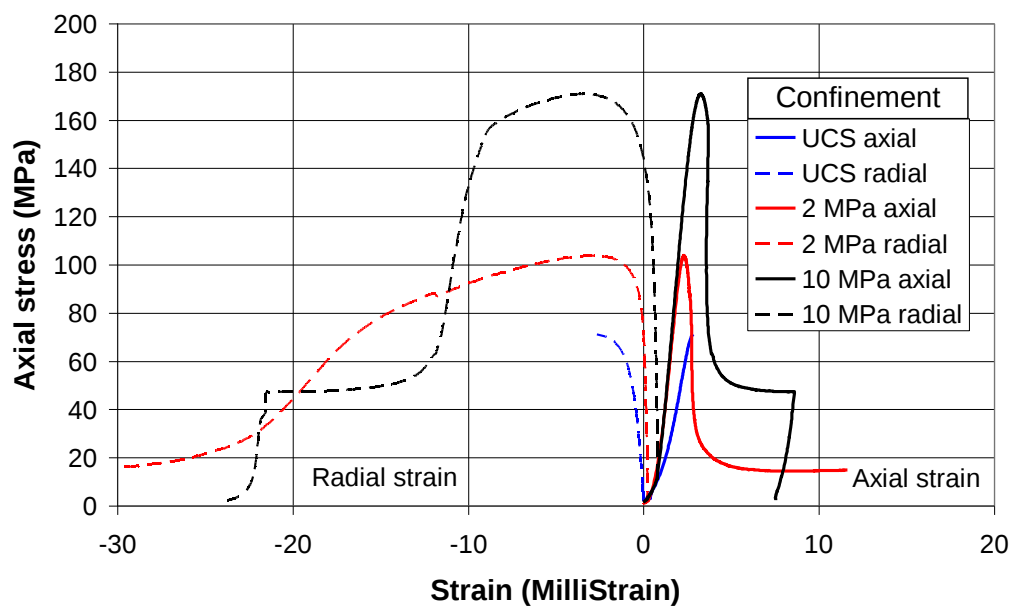


Figure 4-15 Stress-strain behaviour of the Impala site hangingwall pyroxenite

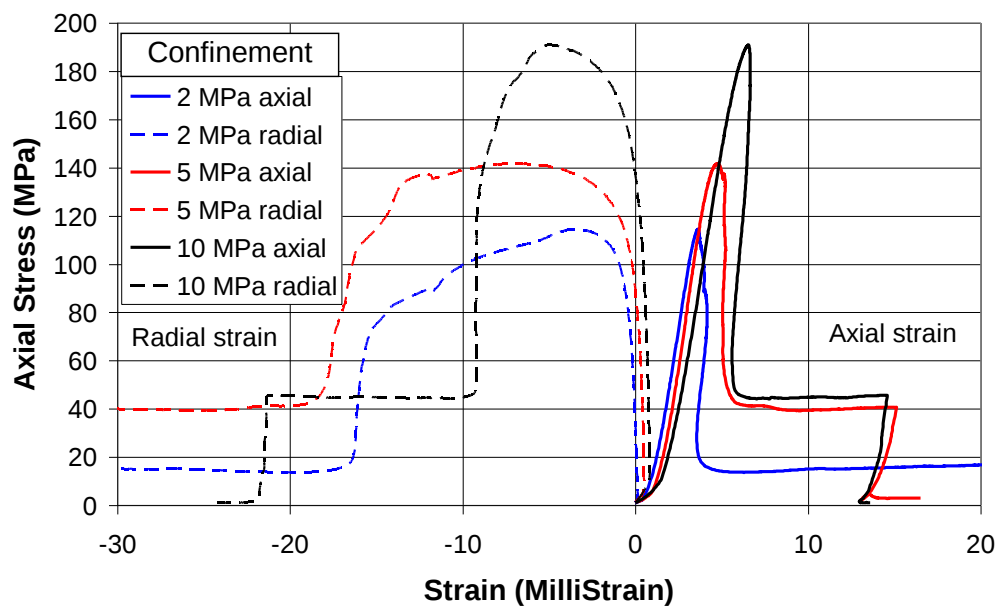


Figure 4-16 Stress-strain behaviour of the Impala site footwall anorthosite

The selected strength parameters for the Mohr-Coulomb strain-softening model were a cohesion of 20 MPa and an internal friction angle of 40°. These parameters resulted in a UCS of 86 MPa, which was similar to the laboratory-determined UCS (Figure 4-15). The results were obtained from models with a high mesh density in which the pillar consisted of 48 square elements across the height of the pillar. The stope span was five times the pillar width (extraction ratio ~ 83%) and the model height was more than eight times the pillar width. A further increase in mesh density did not have a substantial effect on the calculated punch resistance. Two extremes were selected: a relatively brittle material with a cohesion loss of 20 MPa over 25 millistrain (brittle model), and a relatively ductile material with a cohesion loss of 20 MPa over 100 millistrain (ductile model). The more ductile material properties were inferred from underground measurements at the Amandelbult site.

The ultimate punch resistance for comparatively brittle and ductile materials was determined by comparing the strengths of pillar systems where the foundations were not allowed to fail to models where failure occurred in both the pillar and the foundations (Figure 4-17). Failure was thus concentrated in the pillar where the foundations were infinitely strong and the so-called “squat” effects were demonstrated. Suites of pillars with w/h ranging from 0.5 to 10 were used in the

comparison. It was found that punching could initiate at a w/h ratio of 1.2 for more ductile materials, whereas this initiation occurred at a higher w/h in more brittle materials (the Merensky pillars follow the more ductile profile). At smaller w/h ratios, the pillars fail by progressively crushing from the edges towards the core, but in the wider pillars additional fracturing of the hanging- and/or footwall rock is initiated. Figure 4-17 shows that there is a disparity between the strengths of pillars with and without elastic (infinitely strong) foundations. This disparity suggests that punching is initiated once the strength exceeds 250 MPa ($\sim 3 \times$ UCS).

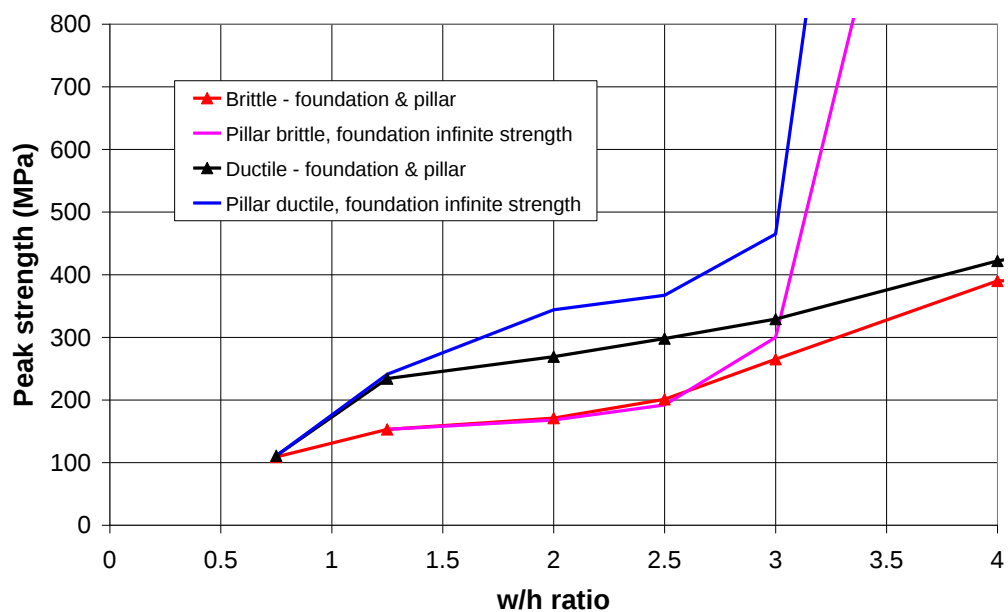


Figure 4-17 FLAC model showing effect of pillar w/h ratio for pillars that are allowed to punch, as well as for pillars that are surrounded by an infinitely strong rock mass

A realistic model of pillar behaviour includes the presence of the hanging- and/or footwall. In such a model the fracturing or damage can expand beyond the pillar itself. This punching phenomenon becomes an important aspect of the failure mechanism of the pillar system, and effectively controls the pillar strength at larger width-to-height ratios.

Fracture localisation is enhanced in the case of a denser mesh, which implies that foundation fracturing occurs more easily in a model with a fine mesh than in a model with a coarse mesh. However, foundation fracturing is not synonymous

with foundation failure. Foundation failure is the final stage, in which vertical punching is accommodated by horizontal dilation. The mesh density may have a different effect on the various pillar failure mechanisms – i.e. fracturing of the pillar itself, fracturing into the foundation and, finally, the dilation and failure of the foundation. This issue requires further investigation. In order to obtain a representative material brittleness as well as a satisfactory correlation between pillar failure and rock mass failure, the combination of mesh density and rate of cohesion softening needs to be calibrated properly. Figure 4-18 shows possible foundation failure distributions for two pillar geometries, with the calibrated mesh densities and cohesion softening rates. It should be noted that the grid may have influenced the development of the vertical cracks.

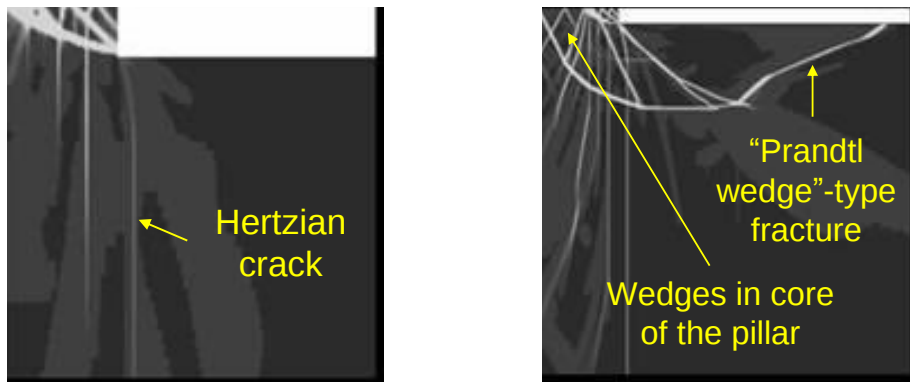


Figure 4-18 FLAC: failure distribution, using dense mesh and ductile material; $w/h = 2.0$ (left) and 5.0 (right) (double symmetry)

It is of interest to note that the pillar with a w/h ratio of 2.0 is completely crushed, with limited failure in the foundation, while the pillar with a w/h ratio of 5.0 shows extensive footwall failure combined with relatively large solid wedges in the core of the pillar. A Prandtl Wedge (Prandtl, 1921), as shown in Figure 4-19, is essentially a slip line (shear fracture) in a granular material that enables the foundation (footwall) material to dilate into the stope, thus accommodating the actual pillar deformation and failure. The numerical models of brittle pillar failure show a more complex arrangement of fractures and the Prandtl wedge does not seem to be reproduced exactly as in Figure 4-19.

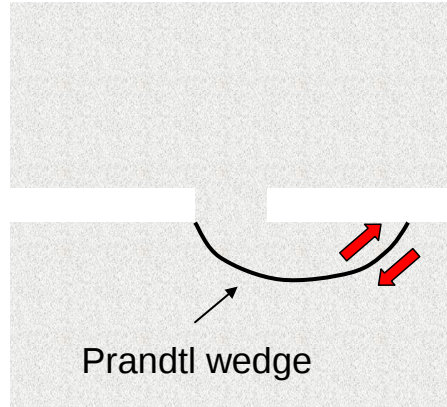


Figure 4-19 Diagram showing a typical Prandtl wedge

Failure of a pillar system, which includes the adjacent footwall and/or hangingwall rock, involves in essence a combination of four mechanisms. First, there is fracturing and crushing of the pillar itself, which can be reproduced under laboratory conditions with unrealistic boundary conditions. Second, there is the tensile fracturing into the surrounding material – the Hertzian crack (Hertz, 1896). Third, wedge formation occurs in the form of shear fractures. The fourth and final mechanism is the horizontal dilation of wedges formed in the foundation, which controls the ultimate resistance against punching. The horizontal resistance against this wedge dilation determines the ultimate punch resistance. The ultimate punch resistance can be expressed as follows:

$$\sigma_p = \frac{1 + \sin \phi}{1 - \sin \phi} UCS_h. \text{ (Ryder and Jager, 2002)} \quad 4-3$$

σ_p = Punch strength

ϕ = Internal friction angle

UCS_h = *In situ* horizontal strength

These last two mechanisms have only been investigated to a very limited extent as far as brittle materials are concerned and references to them are therefore few (Cook et al, 1984; Dede, 1997; Özbay and Ryder, 1990; Wagner and Schümann, 1971; York et al, 1998). It is, however, clear that with the probable exception of very slender pillars pillar failure in hard rock is to a large extent controlled by the

fracture and failure processes in the foundation. For this reason, these processes need to be included in any realistic analysis.

Figure 4-20 shows the effect of deformation on foundation fracturing. In Figure 4-20 (A) the average strain across the pillar was 80 millistrains. When this strain was increased to 125 millistrains, significantly more horizontal fracturing developed below the pillar and stope, even though pillar failure had already taken place. Unfortunately, the two diagrams in Figure 4-20 were derived from separate models with slightly different configurations and are, therefore, not directly comparable. A practical example of foundation fracturing and hangingwall deterioration with strain and time was observed on the down-dip side of Impala P1 (Figure 4-21). Just after pillar failure a closure station was installed adjacent to this side of the pillar. Deterioration of the hangingwall over a six-month period resulted in several minor FOGs even though the mining faces on the up-dip and down-dip sides of the pillar had advanced more than 30 m from the pillar.

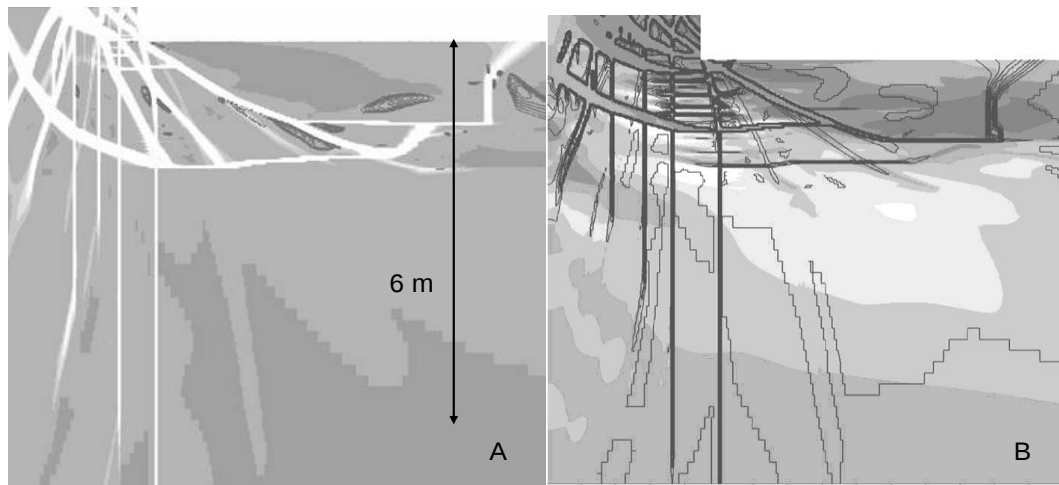


Figure 4-20 FLAC: fracturing in the foundation and pillar after 80 millistrains (white in A) and 125 millistrains (black in B)



Figure 4-21 Impala site: FOGs adjacent to a pillar, resulting from horizontal fracturing in the hangingwall adjacent to the pillar

4.3 Hangingwall investigations

This section deals with the underground measurements of stress and deformation at the three instrumentation sites. Field stress and stress change measurements were conducted above the pillars and panels. The stresses are compared to numerical models, and discussed together with underground measurements of deformation and relevant observations. A detailed description of the underground work is provided in Appendix E.

4.3.1 Shallow depth good rock mass conditions (Amandelbult site)

The mining and pillar configuration at the site (600 m below surface) was typical of a shallow depth, breast layout as described in Chapter 1.3. A plan showing the location of the three closure stations adjacent to Pillar 2 (P2) is presented in Figure 4-22. The red areas in the figure represent the mining that occurred before the instrumentation programme and the subsequent face advances are shown by a series of colours. The grey shows the unmined areas and pillars.

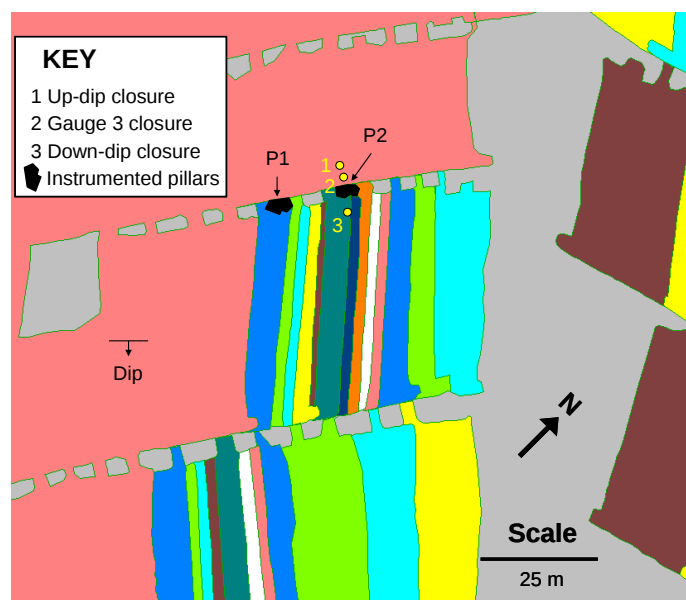


Figure 4-22 Amandelbult site: stope sheet showing the positions of the closure meters around P2

Strain-based stress change measurements were conducted 6.5 m above Pillar 2 (P2 in Figure 4-26) while the face in the down-dip panel was advanced. The strain was converted to stress using the 'matrix' elastic constants (Section 3.6.5). This was necessary since the laboratory UCS test from the position of the stress measurements showed a nonlinear stress-strain relationship. As discussed in Chapter 3.5, the micro-fracturing probably occurred as a result of drilling under high stress conditions, and the *in situ* rock was therefore not micro fractured.

The results of the vertical stress change are compared to an elastic model in Figure 4-23. The figure shows a reasonable correlation between the measurements and the model for a face advance of 7 m beyond the measurement point. The slightly lower stress change predicted by the model is most likely due to the incorrect assessment of the additional stress resulting from mining external to the modelled area. A residual pillar strength of 20 MPa was assumed in the model from the evaluations described in Section 5.6 and this could have been slightly high.

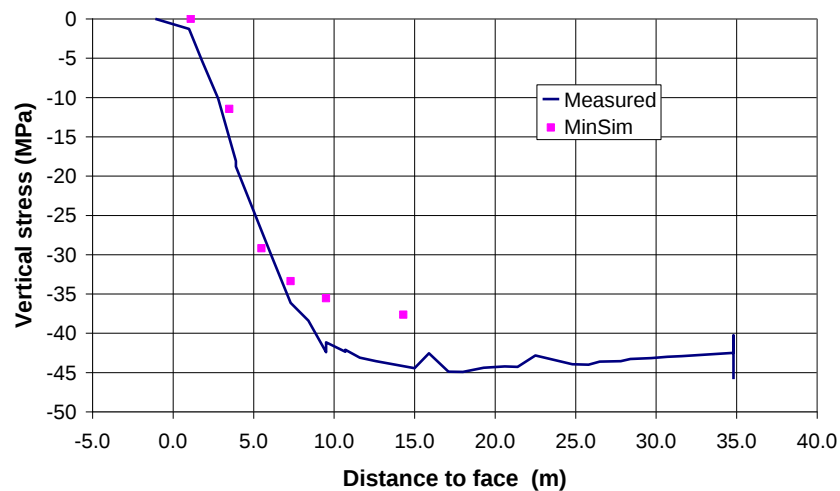


Figure 4-23 Amandelbult site: vertical stress changes at 6.5 m above Pillar 2

The horizontal stress measurements conducted at this point deviated significantly from the elastic model, although the initial trends are the same. Note that the horizontal stress approximately perpendicular to the pillar line showed the greatest deviation, being an increase in compressive stress. Possible reasons for the difference are the influence of the beams that are present above the stope or,

possibly, some effects of fracture development above the pillar during failure. A FLAC model was used to determine the effects of fracturing and foundation damage on horizontal stress and, although the observed fracturing was not the same as in the model, increased horizontal stress was evident in the model (Figure 4-25).

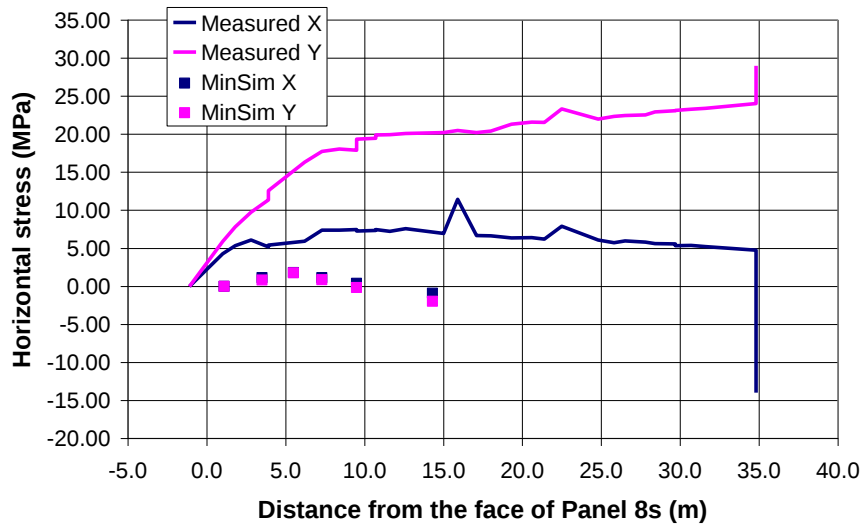


Figure 4-24 Amandelbult site: horizontal stresses at 6.5 m above P2. X-direction along the pillar line, Y-direction perpendicular to the pillar

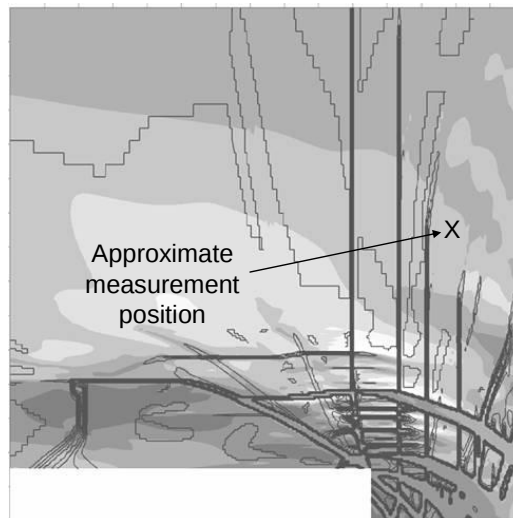


Figure 4-25 FLAC model showing possible fracturing (black) and horizontal stress. The light colours represent relatively high stress conditions. Contours = 25 MPa

During the mining period the stope was dry. However, after an adjacent stope from another raise line had mined through into the instrumented stope some months after completion of the instrumentation site, water was observed dripping from the vertical joints in the hangingwall adjacent to the 15 m-x-15 m stability pillar. As no water was intersected during drilling operations up to the Bastard Reef, it is very likely that joints opened or 'stress-fractures' developed to a height of greater than 20 m above the pillar/stope as a result of the increased pillar stress.

Horizontal field stresses were measured in a vertical borehole drilled into the hangingwall at the centre of panel 12-16W-5E (V2 in Figure 4-26). The measurements were conducted after mining was completed in the stope and the final face positions are shown in Figure 4-26.

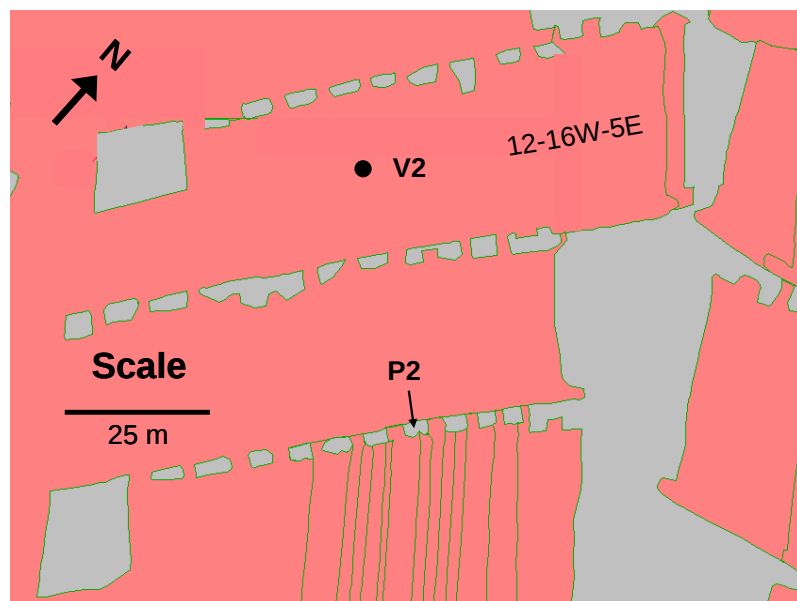


Figure 4-26 Amandelbult site: stope sheet showing the vertical borehole (V2) drilled into the hangingwall

The method employed in evaluating the stresses measured in Borehole V2 is described in Chapter 3.8 and Appendix B, and a description of the site is provided in Appendix E. The results of the investigation are shown in Figure 4-27. The stress profiles suggest a series of beams. However, only one parting plane was observed (with an aperture of between 2 mm and 5 mm) at a height of about 2.5 m (black dashed line in Figure 4-27) and no parting occurred at the base of

the Bastard Reef. The other peaks and troughs in the stress profile all coincided apparently with changes in lithology but no parting planes were observed. The deviations from the MinSim model may be due to the effects of the different elastic properties of adjacent rock types on each other, which the model cannot reproduce. A k-ratio of unity appears applicable to the leuconorites, while the darker rocks (norites, melanorites and pyroxenites) suggest a slightly higher k-ratio of about 1.2.

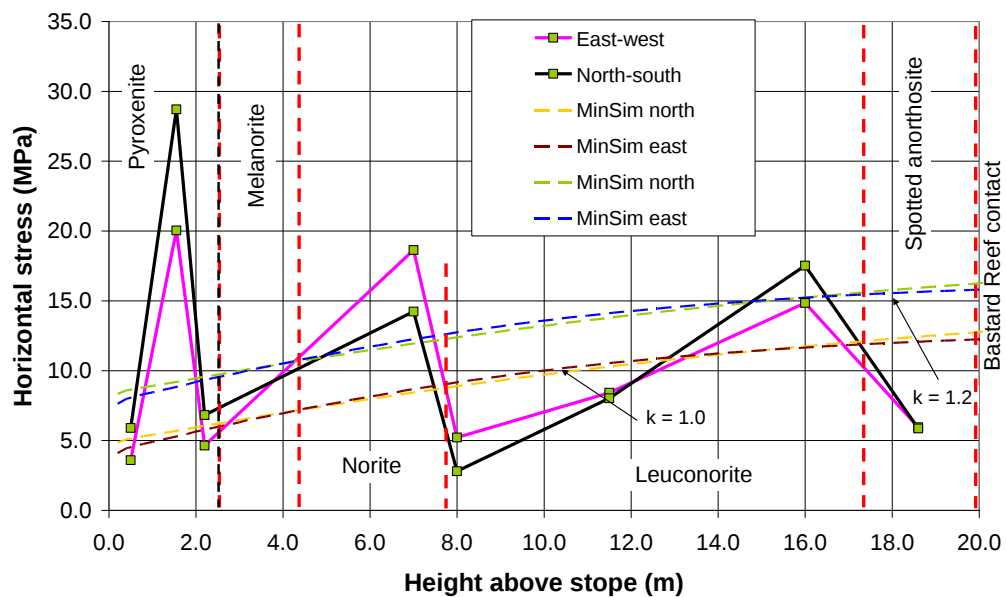


Figure 4-27 Amandelbult site: horizontal stresses in the north and east directions, elastic model dotted. Stratigraphic boundaries marked by red dashed lines and the observed parting shown by a black dashed line

4.3.2 Intermediate depth good rock mass conditions (Impala site)

The vertical- and shallow-dipping boreholes used to conduct borehole camera surveys, extensometer measurements and stress measurements in the hangingwall of the Impala site are shown in Figure 4-28. The face positions at the time of the stress measurements conducted in boreholes V1a and V1b are shown in the figure. CSIRO1 was drilled ahead of both panels and the shallow-dipping

boreholes, including CSIRO2, were drilled when Panel 7s was an isolated panel – i.e. ahead of Panel 8s. The workings are at a depth of 1100 m below surface.

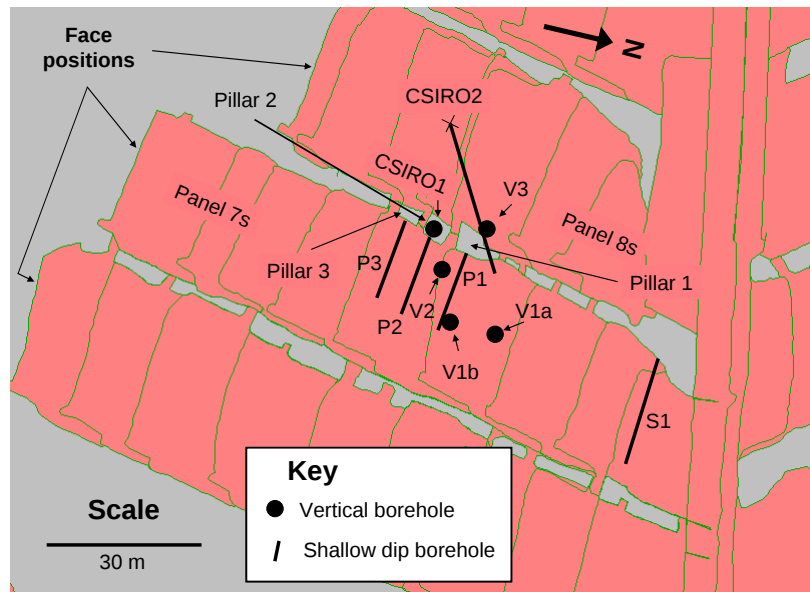


Figure 4-28 Impala site: stope sheet showing the boreholes used to conduct stress measurements, borehole camera surveys and extensometer measurements

An extensometer was installed through the centre of Pillar 2 (CSIRO1 in Figure 4-28) from the haulage below, as shown in Figure 4-29. The aim of the instrumentation was to monitor pillar behaviour and the effects of the pillar on its foundations. The instrument was installed ahead of both the up- and down-dip panels and was intended to monitor deformation before and after pillar failure. Unfortunately, shearing of the borehole occurred soon after pillar failure and the subsequent post-failure results are therefore unreliable.

This section describes the measurements conducted in the hangingwall between the top contact of the pillar and 4.18 m above the pillar. The results are shown together with an elastic model in Figure 4-30. A good comparison between the measurements and the model was achieved for the pre-failure results when the 'matrix' elastic constants (Section 3.6.5) were applied in the model. The comparison suggests that the elastic constants of the unfailed rock mass over the pillar were the same as for the small intact laboratory specimens.

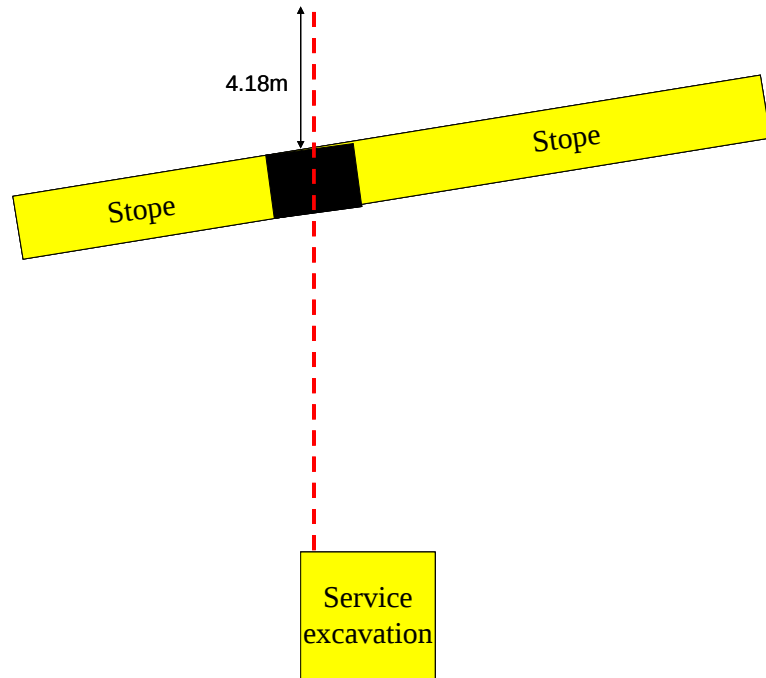


Figure 4-29 Impala site: section along CSIRO1, drilled through the centre of Pillar 2 (not drawn to scale)

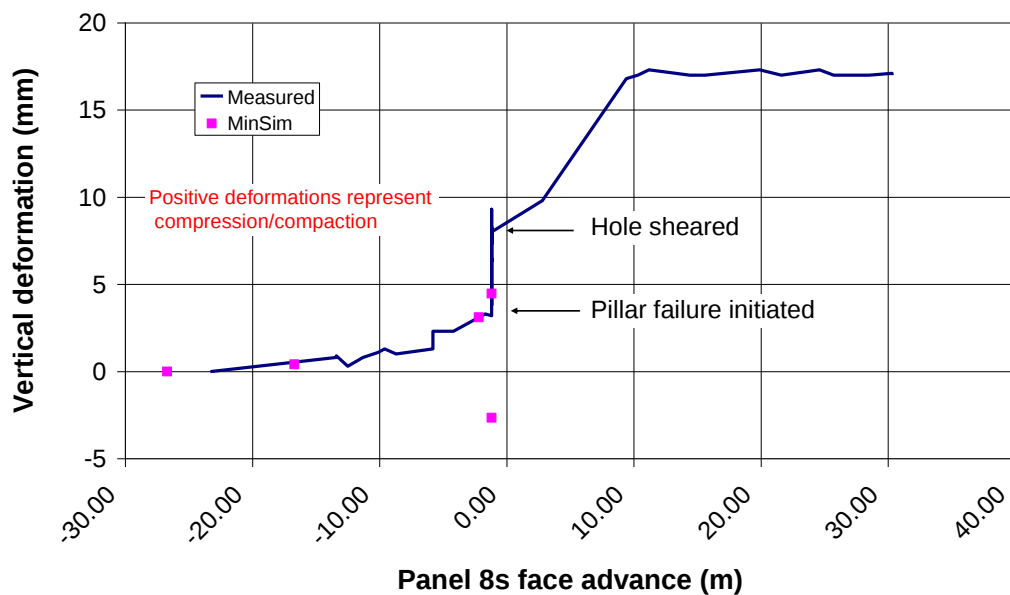


Figure 4-30 Impala site: measured and elastic hangingwall deformations above the centre of Pillar 2, between the pillar top contact and 4.18 m above the pillar. $E = 83$ GPa and $\nu = 0.32$ were used in the model

Reliable post-failure deformations were measured between the points marked as “Pillar failure initiated” and “Hole sheared” in Figure 4-30. The results suggest that the pillar punched initially into the hangingwall. Unfortunately, the exact amount of punching cannot be accurately determined since the results subsequent to the borehole shearing are unreliable. However, at least 4.7 mm of inelastic deformation appears to have occurred before the hole sheared. This punching does not appear to have resulted in significant damage over the stope. No fracturing was observed in the vertical borehole at the centre of Panel 7s and parting did not occur at the base of the Bastard Reef. The cores of eight shallow-dipping boreholes, drilled into the hangingwall during and after mining had been completed in the stope, were inspected for fractures and none were found over the panel. These boreholes covered a wide area of the hangingwall from S1 to P3 (Figure 4-28). However, vertical fractures were observed above the pillars and the following shallow-dipping fractures were observed near the edge of the pillars:

- an open discontinuity at 0.23 m above the hangingwall at a distance of 1.5 m from the pillar edge; and
- a possible fracture plane, which was closed, at 1.58 m above the hangingwall, about 5.5 m down dip of the pillar.

No partings were observed at the base of the Bastard Reef either in V1a (Panel 7S centre, Figure 4-28) or in V2 (5.5 m down dip of Pillar 2, Figure 4-28). An extensometer that was installed in the latter borehole also showed that no inelastic deformations occurred in the hangingwall from the time of installation to the end of the monitoring programme.

During the drilling of a shallow-dipping borehole over Pillar 1 (Figure 4-28), drill water percolated through steeply dipping joints up to a horizontal distance of 2 m from the pillar edge. The height of the borehole was about 1.2 m above the hangingwall when this percolation initiated. Sub-vertical fracturing was observed in the borehole core from the section of the borehole located over the pillar. These fractures were parallel to the pillar line. A horizontal spacing of about 0.1 m was estimated between fractures from the core intersections. Since the borehole was drilled slightly steeper than the reef, the distance between the borehole and pillar increased towards the up-dip side of the pillar. The top of the

vertical fracturing was observed at about 1.2 m above the pillar. It appears, therefore, that vertical fractures formed to a height of 1.2 m above the pillar. Water was able to percolate through these fractures and into the vertical joint. The joint was also obviously slightly open to a distance of 2 m from the pillar edge. Similar fracturing was observed to the same height above Pillar 2. Closely spaced, vertical fractures (cm spacing) could also be observed in the hangingwall-pillar contact of all three pillars where the pillar sidewalls had fallen away (Figure 4-31). It appears that only relatively few fractures penetrated to 1.2 m above the pillars. Figure 4.31 also shows vertical fracturing in the pillar itself. Note that the change in rock type is about the mid-point of the vertical height of the pillar.

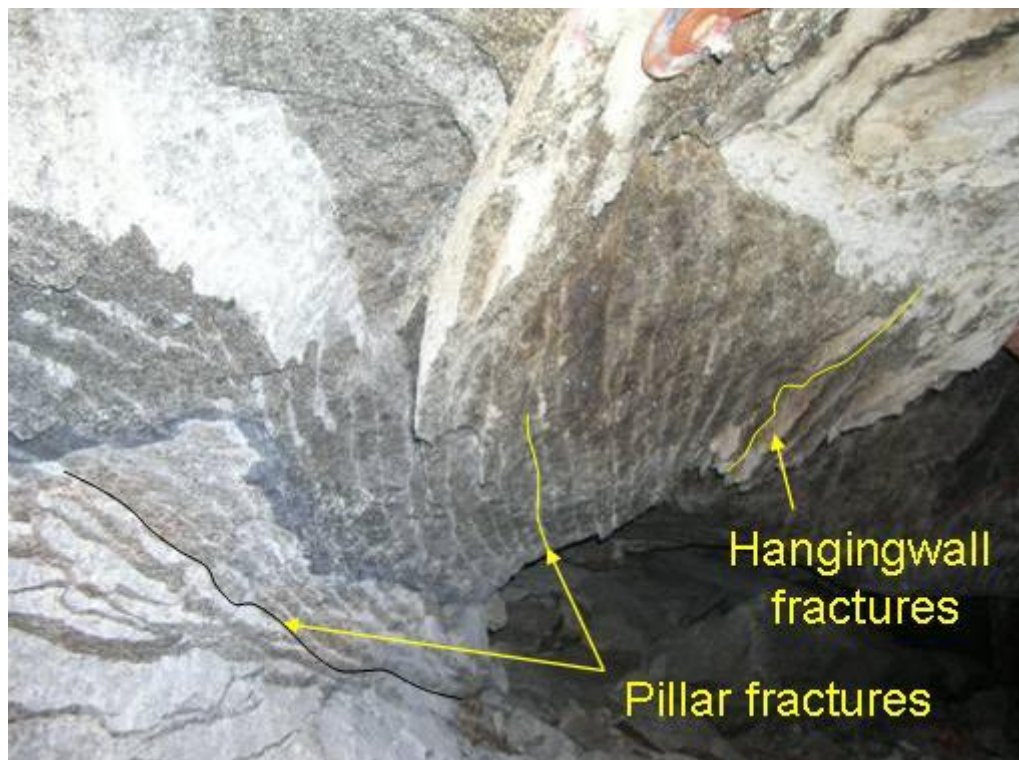


Figure 4-31 Impala site: fracturing in the pillar and hangingwall above a pillar, exposed when the pillar sidewall fell away

An extensometer was installed in a shallow-dipping borehole drilled over Pillar 1 and above Panel 8s (CSIRO2 in Figure 4-28). A section along this borehole is shown in Figure 4-32. Anchors were positioned to measure the horizontal deformations immediate above the pillar and in the hangingwall above Panel 8s during the mining of Panel 8s. The measurements over the pillar and the 8s panel

were compared to an elastic model and are shown in Figure 4-33 and Figure 4-34 respectively. The 'matrix' elastic constants of $E = 83 \text{ GPa}$ and $\nu = 0.32$ were again assumed in the model (Section 3.6.5).

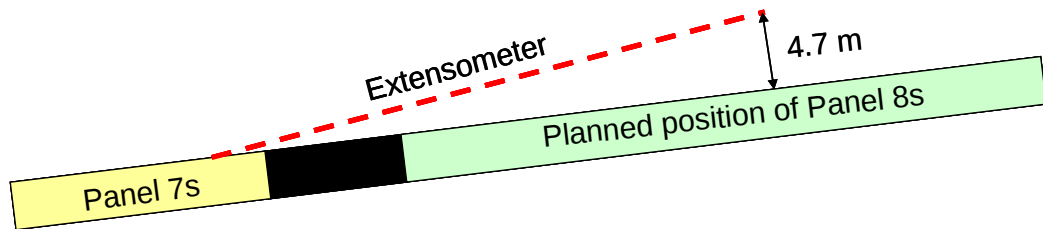


Figure 4-32 Impala site: section along the shallow-dipping extensometer borehole, drilled over the top of Pillar 1 (not drawn to scale)

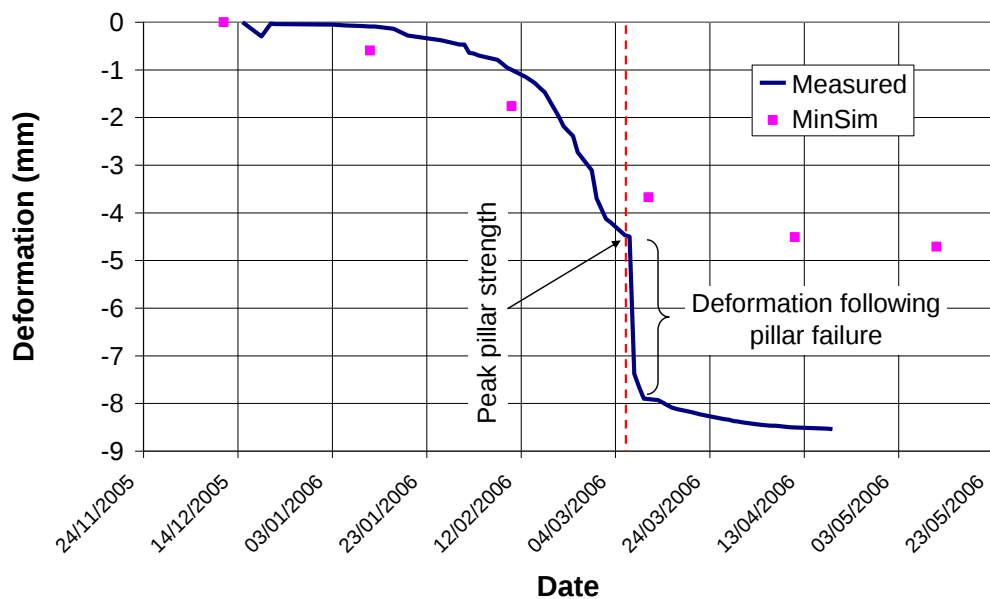


Figure 4-33 Impala site: horizontal dilation in the hangingwall across the top of Pillar 1, about 0.9 m above the pillar. Compression is positive

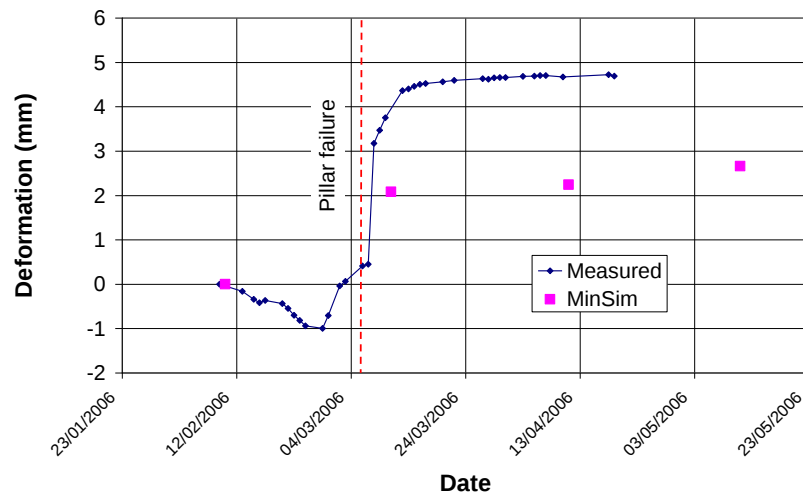


Figure 4-34 Impala site: deformation along a shallow-dipping borehole, between 1.5 m and 5.5 m above Panel 8s. Measurements performed in Borehole CSIRO2 over a length of 19.6 m. Compression is positive

The deformations over the pillar that took place ahead of the Panel 8s face were approximately elastic (Figure 4-33), i.e. the elastic numerical model approximated the measured deformations. A better fit between the measured and elastic deformations might have been achieved if the locations of the lagging siding (between the ASG and the pillars) were known for each mining step. The siding effectively reduced the width of the pillar from 5 m to about 3 m and its advancement was not regular as assumed in the model.

It is clear from the measurements in Figure 4-33 that once pillar failure occurred (about 2 m behind the face), significant tensile, inelastic deformations were induced over the pillar; i.e. the deformations were more tensile than the elastic model. Figure 4-34 shows corresponding compressive, inelastic deformations over the stope during the same period. The negative inelastic deformations over the pillar are tensile strains, probably resulting from pillar dilation and resultant formation of vertical fractures in the hangingwall foundation. However, the opening of micro fractures (Chapter 3) may have occurred when the stress dropped and this would have contributed to the deformations by altering the effective modulus.

The inelastic deformations between the pillar and the centre of the panel (Figure 4-34) are positive, which means that the reference points moved closer to one another by about 2.5 mm over a length of 19.6 m. This suggests an average induced compressive stress in the horizontal direction of between 2 MPa and 11 MPa if the nonlinear- and linear material properties are assumed, respectively. The induced compressive stress above the panel is probably partly the result of fracture development and damage over the pillars.

A 3D strain cell was installed at the end of borehole CSIRO2 (Figure 4-28 and Figure 4-32) – i.e. in the same borehole as the extensometer – to measure stress change over the centre of Panel 8s with face advance. Stress change was thus monitored ahead of and behind the face from a height of 4.7 m above the panel. Figure 4-35 shows a comparison between the measured horizontal stress in the dip direction (evaluated using the ‘matrix’ elastic constants of $E = 83$ GPa and $\nu = 0.32$) and an elastic model. The measurements show a significantly greater increase in compressive stress than the model during pillar failure. At least some of this induced stress may have been the result of dilation and fracturing over the pillar.

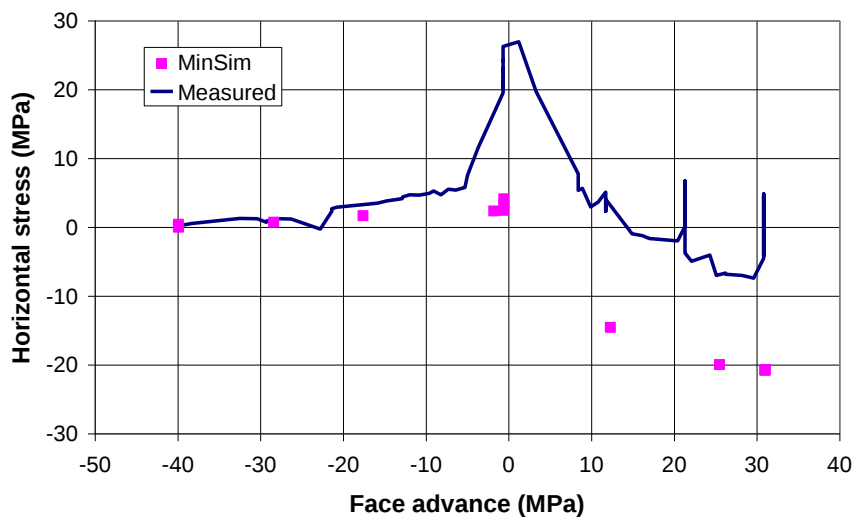


Figure 4-35 Impala site: horizontal stress change in the dip direction, 4.7 m above the centre of Panel 8s in Borehole CSIRO2. Negative face positions are behind the cell. Compression is positive

The vertical stress measurements in Borehole CSIRO2 are shown together with the elastic results as a function of face advance in Figure 4-36. Again, the measurements were evaluated using the 'matrix' constants ($E = 83 \text{ GPa}$ and $\nu = 0.32$). The comparison suggests that the rock mass behaved in an elastic manner at the position of the instrument until the face was about 8 m ahead of the instrument. Further face advances appear to have resulted in the development of micro fractures in the rock mass and a change in the effective modulus at the position of the cell, as discussed in Section 3.7. The micro-fracturing of the rock mass appears to have occurred here, and elsewhere where vertical stress change measurements were made at the site, when the vertical stress levels dropped below about 10 MPa (Section 3.7).

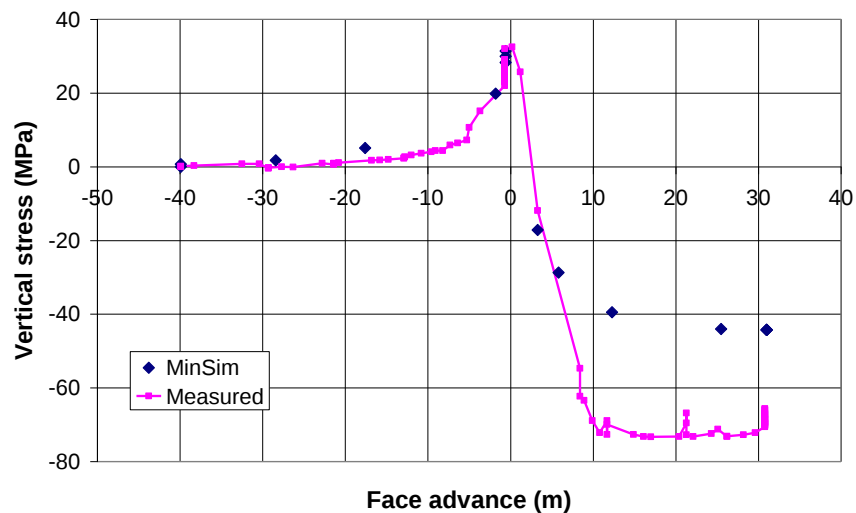


Figure 4-36 Impala site: vertical stress change 4.7 m above the centre of Panel 8s in Borehole CSIRO2. Negative face positions are behind the cell

A comparison between the measured and elastic horizontal stress in the strike direction for CSIRO2 (North in Figure 4-28) is shown in Figure 4-37. A significant deviation from the elastic model appears to have occurred from a face position of about 5 m behind the cell position. The sudden large increase in compressive stress could not be explained by foundation damage above pillars. A 2D DIGS (Napier and Hildyard, 1992) model was run to observe the possible effects of the presence of a horizontal discontinuity. This model showed that high horizontal compressive stresses are present in the strata just above a discontinuity. In

essence the presence of a stope-parallel discontinuity artificially creates a new 'hangingwall' from the top surface of the discontinuity. The measured profile shown in Figure 4-37 is matched fairly well by the modelling results if a horizontal discontinuity is assumed to be present just below the measurement point. Although no fracturing occurred above the 7s panel, such features were observed in a brow created by a FOG that occurred at the stopping position of the 8s face (Figure 4-28). This location was about 30 m from the measurement point. A photograph showing these fractures is included in Figure 4-38.

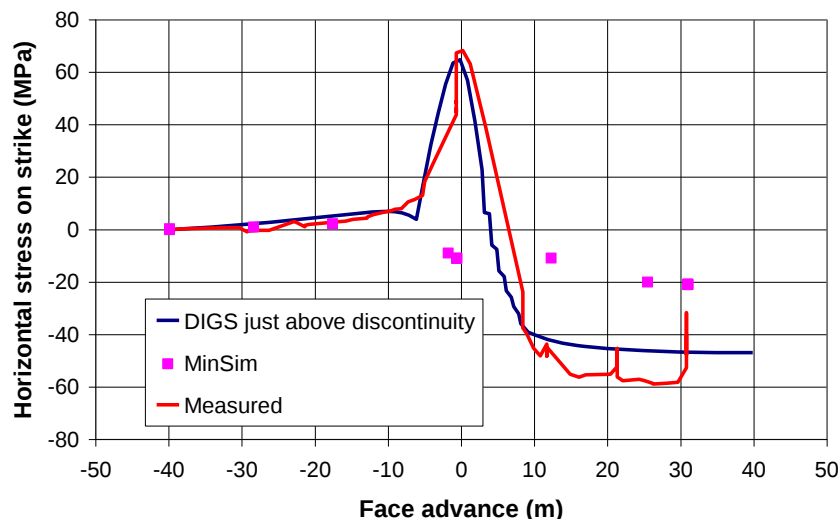


Figure 4-37 Impala site: horizontal stress change in the strike direction, 4.7 m above the centre of Panel 8s in Borehole CSIRO2. Negative face positions are behind the cell



Figure 4-38 Impala site: horizontal fracture planes observed in the brow at the edge of the FOG in the face of Panel 8s

The modelling results shown in Figure 4-37 assumed a single discontinuity, just below the measurement point, with a friction angle of 10° . A higher friction angle results in a lower peak and a larger drop in stress just ahead of the face. Unfortunately no camera surveys were conducted in Panel 8s to determine the existence of such (a) plane(s) at the location of the cell. The panel was, however, prematurely stopped due to poor hangingwall conditions and a FOG at the face. It is postulated that horizontal fractures developed over Panel 8s because of the higher-than-normal field stresses associated with a remnant condition. The higher stress and additional sag resulting from a much larger stope (now consisting of many panels) may have been sufficient to cause the formation of the shallow-dipping fracture thought to be responsible for the additional horizontal stress in the strike direction.

If the advancing fracture and associated shear stress are the reason for the relatively high horizontal stress measured in the strike direction, the stress change in the dip direction would not be directly affected by the assumed shear stress. However, the Poisson effect could partially explain why the measured stress is higher than the modelled stress shown in Figure 4-35. The full explanation is probably a combination of stress generation due to fracturing and damage above the pillars and the Poisson effect. Interestingly, the slopes of the unload portion of the measured and modelled curves, shown in Figure 4-35, are parallel, with an almost constant difference of about 15 MPa between the curves. This comparison suggests that the measurements were dominated by an elastic response after the face had passed under the instrument, and probably no further fracturing occurred at this point in the 8s hangingwall or above the pillars.

Two series of horizontal stress measurements were made at various heights above the centre of Panel 7s in separate boreholes V1a and V1b (Figure 4-28). Borehole V1a (Vert1) was located in a pothole (Figure 4-39) and collared in spotted anorthosite, whereas V1b (Vert1b) collared in normal hangingwall pyroxenite. The first two measurements in this borehole were made in pyroxenite and norite rock types, respectively.

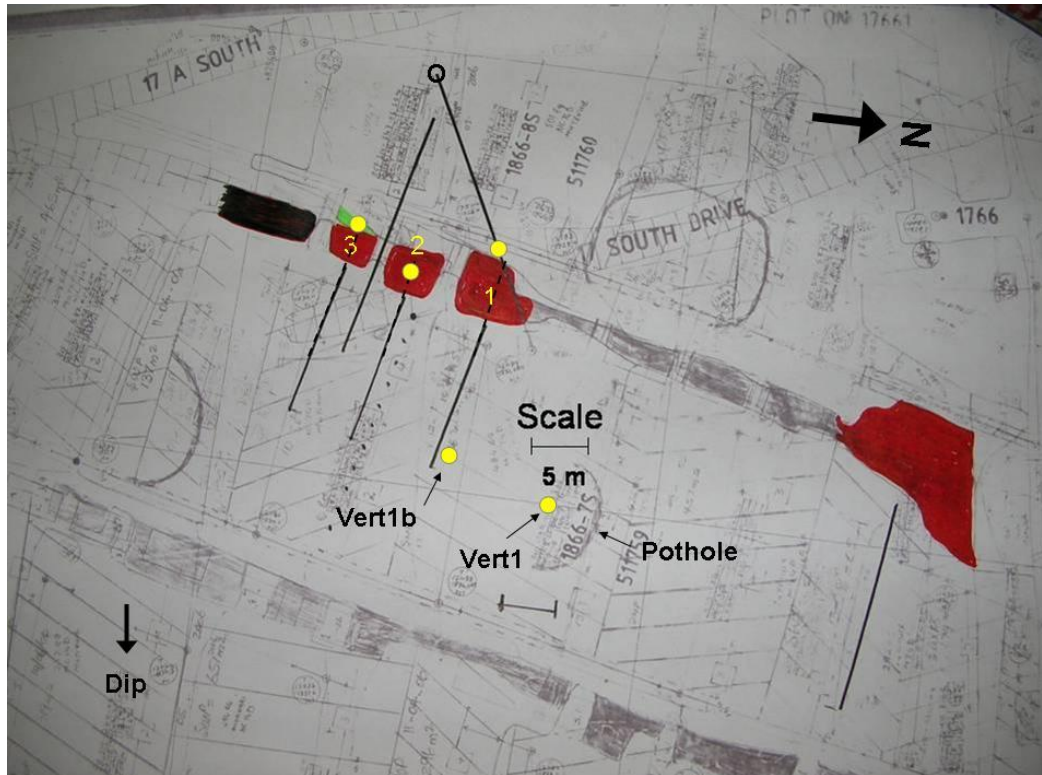


Figure 4-39 Impala site: stope sheet showing boreholes Vert1a and Vert1b and the pothole at the Impala site

The stress measurements in V1a and V1b are compared to an elastic model with a k-ratio of unity, as shown in Figure 4-40. A generally good correlation was observed between the measurements and the model when a residual strength of 30 MPa was assumed for the residual strength of the pillars in the model. (This pillar residual strength was confirmed by the evaluations described in Section 5.6.) The model assumed the virgin horizontal stresses to be the same in all directions, which was probably not true underground. This assumption may have contributed to the small differences between the model and the measurements. In addition, the model did not account for the influence of the fracturing above the pillars (Sigma 2 was measured in the dip direction in Figure 4-40). Both the results shown in Figure 4-40 and the horizontal extensometer over Panel 8s in Figure 4-34 suggest that the stress alteration above the centre of the panel due to fracturing above the pillars may have been small and is likely to have affected only the immediate stope hangingwall.

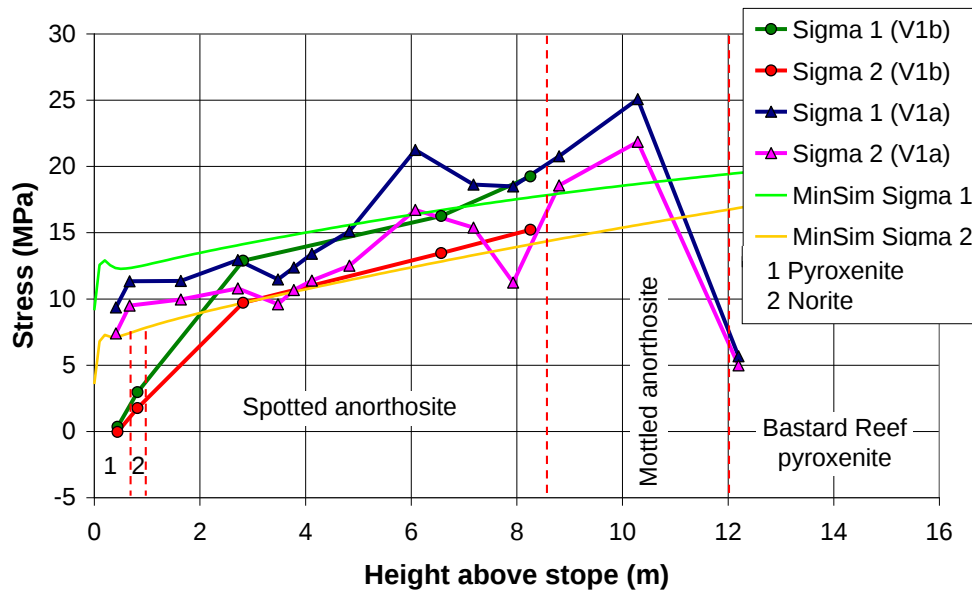


Figure 4-40 Impala site: horizontal stress with height above the stope. A pillar residual strength of 30 MPa and a k-ratio of unity were assumed in the models

The effect of the 9.5 m-wide pothole on stress was evident up to a height about 3 m above the stope (Figure 4-40). Above this height only minor differences were observed between the two boreholes. The shaft geologist confirmed that small potholes in the area generally only affected the stress condition of a relatively small height of rock (Jenkinson, 2006). However, a pink colouration was observed in some of the spotted and mottled anorthosites to a height of about 10 m in Borehole V1a, which were not observed in V1b. This pink colouration in the anorthosites is unusual on the mine and has only been observed in potholes where iron enrichment has taken place (Jenkinson, 2006). The peak stresses at 6 m and 10 m in Borehole V1a (pothole) coincided with a pink spotted and mottled anorthosite respectively. The drop in stress at 8 m in Borehole V1a occurred where the rock changed from pink to white spotted anorthosite.

The generally good correlation between the stress measurements and the elastic results for a k-ratio of unity suggest that no partings were present above the panel up to the base of the Bastard Reef. This suggestion was confirmed by the borehole camera survey, and the contact at the base of the Bastard Reef was also observed to be intact. However, the large drop in stress in the Bastard Reef pyroxenite suggests some or all of the following:

- an interaction between materials with different Young's moduli was measured;
- some sort of beam action occurred; and/or
- a different k-ratio is resident in this rock type.

Since no parting was observed in the borehole camera surveys, the low horizontal stress condition tends to indicate a lower k-ratio in the Bastard pyroxenites or an interaction between materials with different Young's moduli. The pyroxenites and melanorites above the Merensky Reef also showed a comparatively lower horizontal stress condition. These two pyroxenite measurements correlated with a k-ratio of 0.6, as shown in Figure 4-41. The reasonable correlation of the anorthosites and pyroxenites with k-ratios of unity and 0.6, respectively, suggests that the pyroxenites may be in a lower horizontal, virgin stress state than the anorthosites. However, the measurements in the pyroxenites were both close to a change in lithology, and the stress condition at the measurement points may have been influenced by the lower Young's modulus of the adjacent anorthosite lithology.

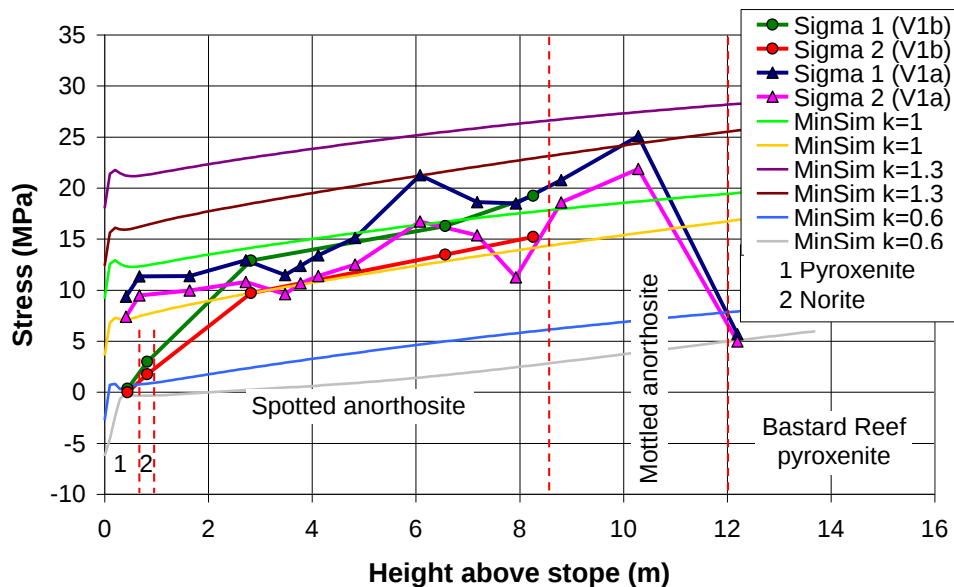


Figure 4-41 Impala site: horizontal stress with height above the slope. A pillar residual strength of 30 MPa was assumed in the models

The normally accepted k-ratio for the area was determined from strain-gauge-based stress measurements conducted elsewhere on the shaft to be 1.3. For completeness, the elastic stress profiles for this k-ratio are also included in Figure 4-41. The reason for the lower-than-expected horizontal stress in the anorthosites is not clear. However, the method of stress evaluation in nonlinear rock masses (Section 3.8) requires confirmation, and an improved system may result in slight differences to the quoted stresses. The actual stresses may, therefore, have been slightly higher than quoted. However, a reasonable correlation between the measured and elastic stresses was shown in the pink anorthosites (6 m and 10.5 m above the stope), when a k-ratio of 1.3 was assumed.

The hangingwall conditions at the Impala site were stable, even though there was evidence that the rock mass had become nonlinear in the places where the vertical stress had dropped below about 10 MPa.

Stress change measurements were taken over the three monitored pillars shown in Figure 4-28. 2D straincells (doorstoppers) were installed at 5.28 m and 3.23 m above the down-dip edges of Pillar 1 and Pillar 3 in boreholes P1 and P3, respectively (Figure 4-28). A 3D strain cell was installed at 4.2 m above Pillar 2, CSIRO1 in Figure 4-28. The measurements were evaluated using the 'matrix' elastic constants and compared to elastic models. The pre-failure vertical stresses above the three monitored pillars all correlate well with the elastic models (Figure 4-43, Figure 4-42 and Figure 4-44).

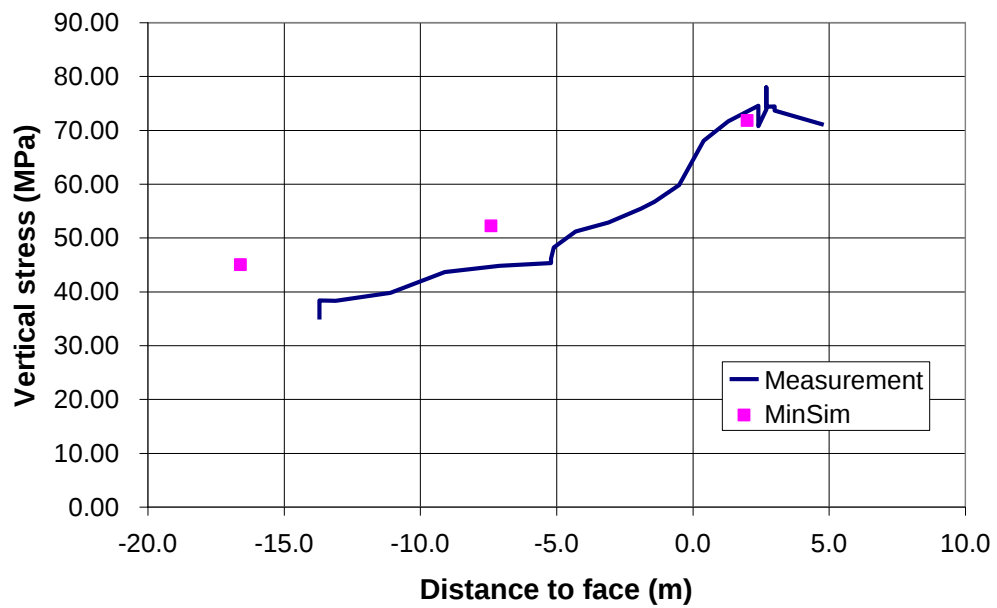


Figure 4-42 Impala site: vertical stress change 5.28 m above the pillar in Borehole P1. Negative face positions are behind the cell

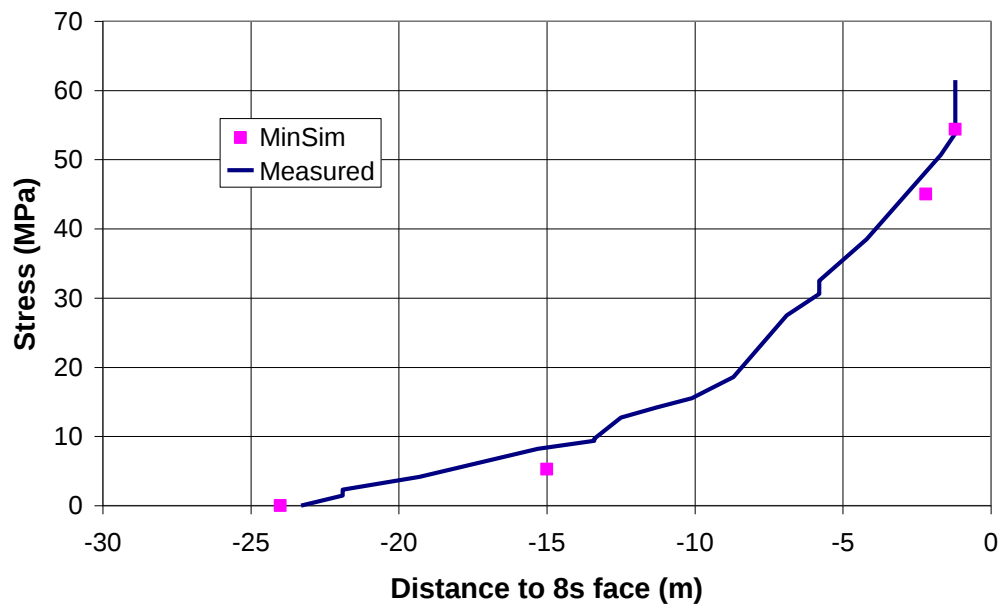


Figure 4-43 Impala site: vertical stress change 4.2 m above Pillar 2 in Borehole CSIRO1. Negative face positions are behind the cell

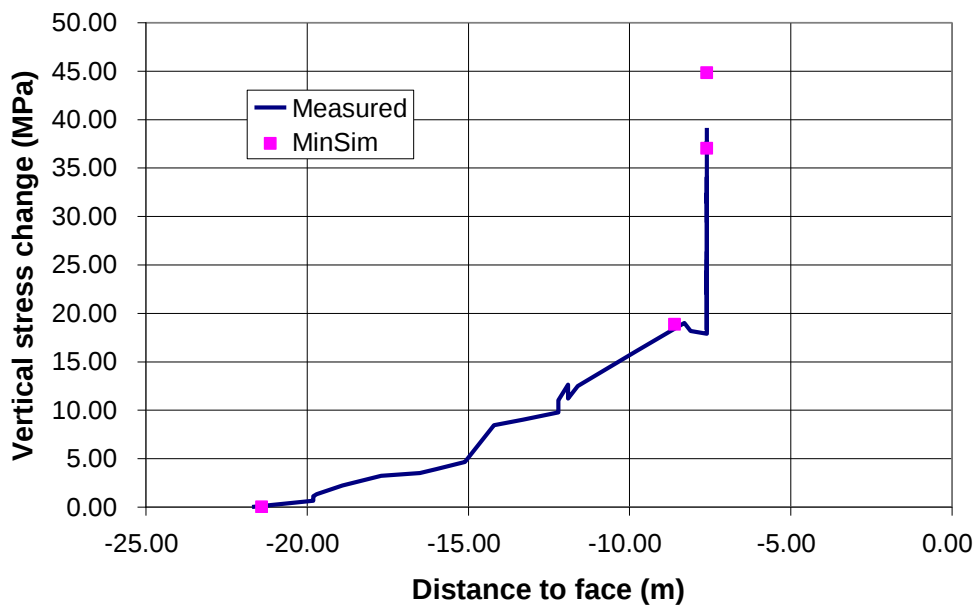


Figure 4-44 Impala site: vertical stress change 3.23 m above the pillar in Borehole P3. Negative face positions are behind the cell

The horizontal stresses measured in the strike direction above pillars 1 and 3 also correlated well with the elastic model (Figure 4-45 and Figure 4-46). However, the horizontal stresses measured above the centre of Pillar 2 were significantly different from the elastic model in both the dip and strike directions (Figure 4-47). The reason for this deviation may be explained by the effects of foundation fracturing and the stress state above the adjacent pillars not being reflected properly in the model. Note that the measured stress increased, while the general trend in the model was a decrease. This result was similar to the horizontal measurements above Pillar 2 at the Amandelbult site.

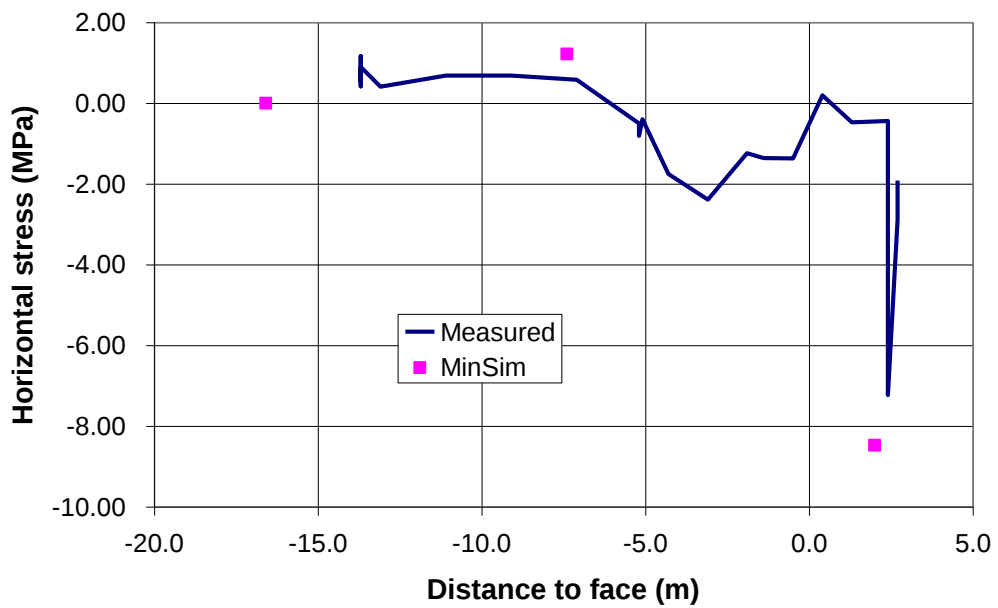


Figure 4-45 Impala site: horizontal stress change in the strike direction, 5.28 m above the pillar in Borehole P1. Negative face positions are behind the cell

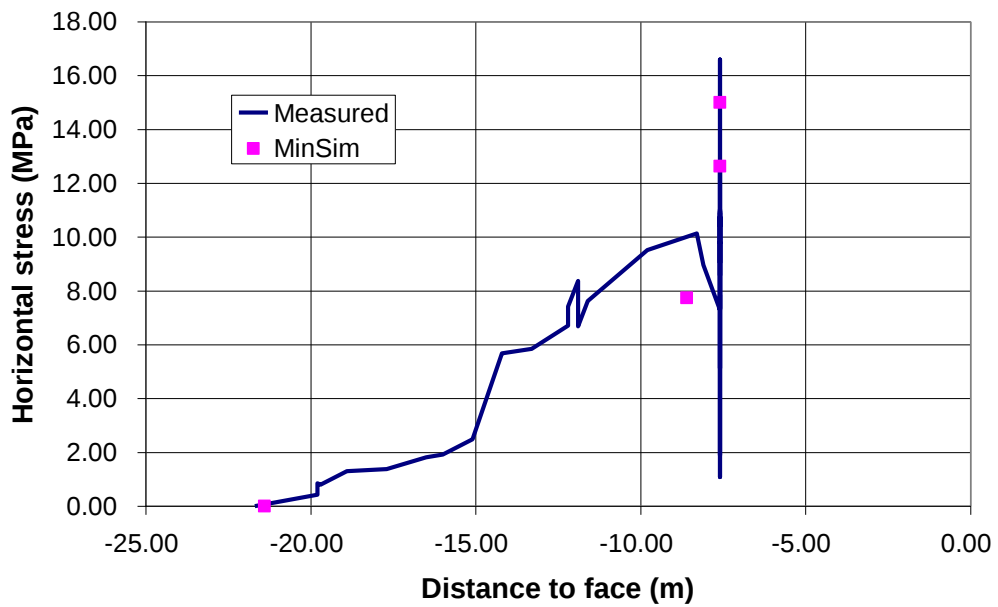


Figure 4-46 Impala site: horizontal stress change in the strike direction, 3.23 m above the pillar in Borehole P3. Negative face positions are behind the cell



Figure 4-47 Impala site: horizontal stress change 4.2 m above Pillar 2 in Borehole CSIRO1. Negative face positions are behind the cell

A FLAC model of a pillar with a w/h ratio of 3 and including foundations (Figure 4-48) was run to determine the effects of the fracturing above a pillar on horizontal stress. The parameters used in the model are described in Section 4.2.

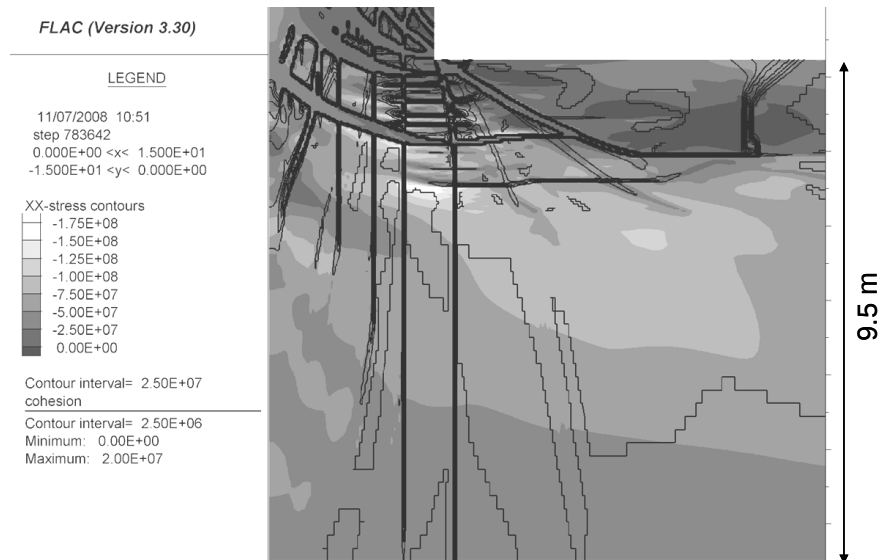


Figure 4-48 FLAC model showing possible fracturing (black) and horizontal stress. Stress expressed in Pa and a negative sign represents compressive stress

The light-grey colours in the model suggest that significant horizontal stress can be generated above and adjacent to a pillar as a result of fracturing and foundation damage. More work should be done to quantify the effects of foundation fracturing and damage on horizontal stress and panel behaviour.

While beams did not form above Panel 7s at the Impala site, there is evidence to suggest that beams could have developed above Panel 8s. For this reason, the horizontal stress change measurement over Pillar 2 might also have been affected by beam formation.

4.3.3 Intermediate-depth poor rock mass conditions (Union Spud shaft site)

Hangingwall behaviour was studied at two similar sites approximately 0.5 km apart on strike. Owing to the poor hangingwall conditions large areas were unmined and the stope spans were smaller than at the other instrumentation sites. At Site 1 (Figure 4-49), vertical- and shallow-dipping boreholes were drilled for borehole camera surveys, extensometers and stress measurements. A series of horizontal stress measurements was drilled from the centre of a stope at the second site (Site 2 in Figure 4-50). Both sites were located at depths of about 1400 m below surface.

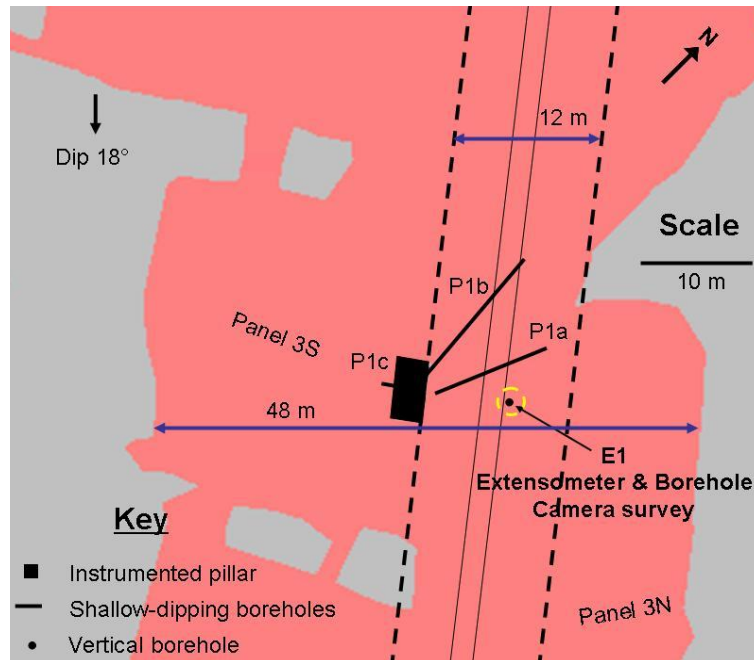


Figure 4-49 Union Site 1: stope sheet showing the boreholes used for the extensometers, borehole camera surveys and the stress measurements

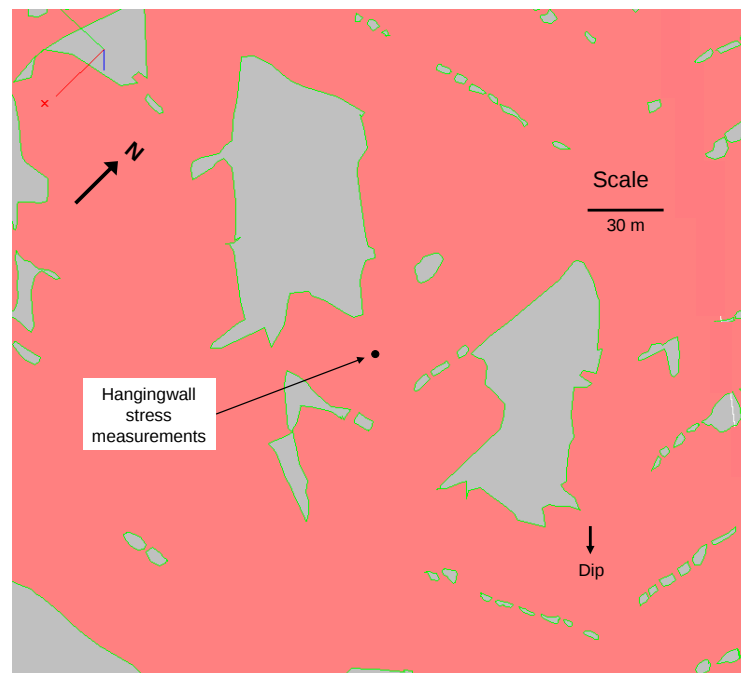


Figure 4-50 Union Site 2: stope sheet showing the location of the horizontal stress measurements conducted in a vertical borehole drilled up into the hangingwall

With the exception of the strain cell in Borehole P1c, all the instrumentation at Site 1 (Figure 4-49) was installed before the mining of Panels 3s and 3n – i.e. when the strike span was only 12 m, as shown by the dashed lines in Figure 4-49. Borehole camera surveys were also conducted before and after Panels 3s and 3n were mined.

The borehole camera surveys showed that significant fracturing or opening of shallow-dipping discontinuities occurred in the hangingwall when the span was increased from 12 m to 48 m (Figure 4-49). A detailed list of observed fractures and geological discontinuities has been documented in Appendix E. Photographs of the core from the extensometer boreholes are also provided in this Appendix. Open shallow-dipping discontinuities were observed up to a height of 3.2 m in the extensometer borehole at the initial 12 m span. This was significantly greater than the theoretical vertical tensile zone of 0.4 m, as determined by the analytical solution for an isolated panel (Ryder and Jager, 2002). Also, open steeply dipping discontinuities were observed up to a height of 11 m, and the elastic MinSim model only shows a horizontal tensile zone up to 1 m when a k-ratio of 0.5 is assumed. An explanation for the difference between the observations and theory is suggested in Section 4.1.2.1 and explained in Appendix D. The abutment stresses may also have influenced conditions by facilitating shearing on the steeply-dipping structures. This, in turn, may also have influenced the shallow-dipping discontinuities. However, no evidence of shearing was observed in the panel or borehole.

After the initial span was increased to 48 m, open shallow-dipping discontinuities were observed up to a height of 12.8 m above the stope. Significantly open discontinuities (both shallow- and steeply dipping) were observed up to a height of 3.4 m above the stope. The theoretical horizontal tensile zone was shown by a MinSim model to be at a height of about 5.8 m (assuming a k-ratio of 0.5). A theoretical vertical tensile zone of 4 m was calculated by an analytical solution provided by Ryder et al (2005) (Equation 4-4), for a panel bounded by ‘crush’ pillars.

$$z_0 \approx \left(\frac{\pi \gamma}{4 \left(\frac{\sigma_r w_c L_c}{C_c} \right)} \right)^{\frac{1}{2}} L^2$$

Where: γ = density (~0.03 MPa/m)
 σ_r = Residual pillar strength (MPa)
 w_c = Pillar width (m)
 L_c = Pillar length (m)
 C_c = Pillar length and holing (m)

The evaluation assumed a span of 20 m, to account for the distance between the dip-'crush' pillar and the north-eastern face (Figure 4-49). A residual pillar stress of 15 MPa (Section 5.6) was also assumed.

Interestingly, the open shallow-dipping discontinuities were all located in the pyroxenite at a span of 12 m. In the second survey (after the span had been increased to 48 m) the comparatively large aperture discontinuities were still confined to this lithology. The greater-than-theoretical extent of open steeply dipping discontinuities suggests that the stress condition resulting from the presence of the many pre-existing discontinuities was too complex for MinSim to model. It is possible that the north face damaged the hangingwall near the surveyed borehole during the mining of Panel 3s. Stress change above the abutment during the mining of Panel 3s may also have resulted in fracture development or opening of joints in the hangingwall. Evidence of horizontal fracturing or opening of joints was shown by the additional shallow-dipping discontinuities that were observed in the second survey but not in the first. Development of horizontal fractures in the pyroxenites has been observed in other investigations at Union Section (Watson, 1996). Open fractures were not observed higher than 13 m above the stope, where a particularly competent layer was observed in the borehole core. There was also no movement at the base of the Bastard Reef about 20.3 m above the stope.

A single-anchor extensometer was installed in Borehole E1 (Figure 4-49) to measure the deformations that occurred in the hangingwall up to 21 m above the

stope. The anchor was installed above the base of the Bastard Reef to include any parting that may occur at this point. The borehole was located 6.3 m from the edge of the instrumented pillar and 5.7 m from the north abutment (Figure 4-49). The measurements are shown together with the results of a similar extensometer in the footwall and an elastic model in Figure 4-51. The model was constructed assuming average elastic constants that were determined from laboratory tests ($E = 100 \text{ GPa}$ and $\nu = 0.32$). The difference between the elastic model and the measurements indicates the possible extent of the observed inelastic behaviour.

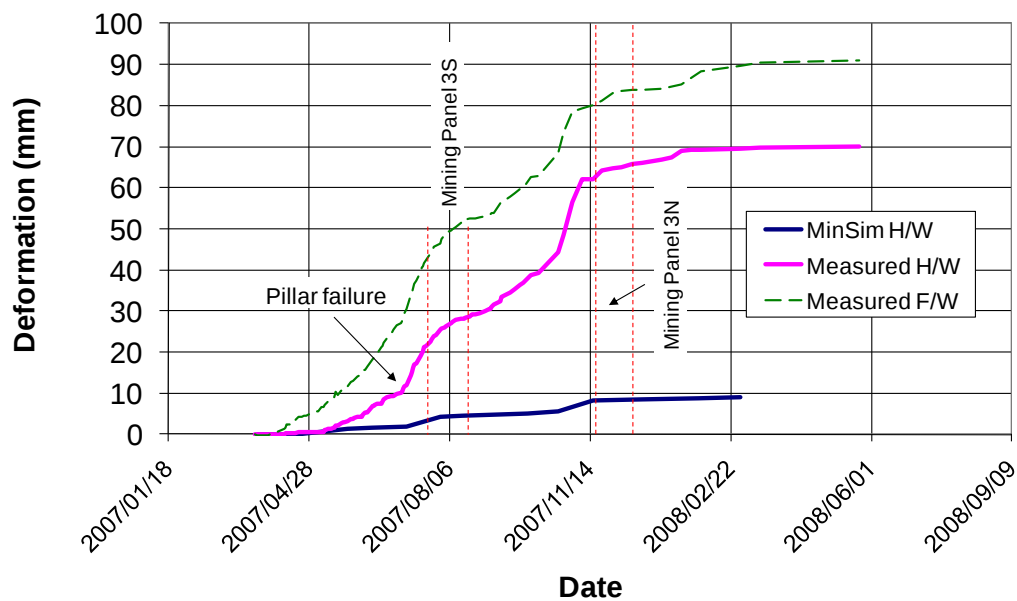


Figure 4-51 Union Site 1: extensometer results. The elastic model assumes $E = 100 \text{ GPa}$ and $\nu = 0.32$

Figure 4-51 suggests that significant inelastic deformations occurred in both the hangingwall and footwall, particularly during the mining of the 3s and 3n panels (Figure 4-51). Generally, though, more deformation was measured in the footwall than in the hangingwall in the ratio of 0.44:0.56 of the total deformation for the hangingwall and footwall, respectively. However, this trend reversed during and just after the mining of Panel 3n, and comparatively more deformation occurred in the hangingwall during this period. The additional deformation is suspected to be the result of gravity on loose blocks in the hangingwall, which were previously supported by the abutment prior to the mining of the panel.

2D strain cells were installed at the end of boreholes P1a and P1b (Figure 4-49). These shallow-dipping boreholes were drilled in the original 12-m wide span and dipped in the opposite direction to the reef. The P1a cell was installed about 3.4 m above the edge of the abutment/pillar, while P1b was installed about 5.3 m over the centre of the proposed pillar. The orientations of the boreholes were such that P1a and P1b measured the horizontal stress parallel to- and perpendicular to the long axis of the proposed pillar, respectively. The measurements were evaluated using the 'matrix' elastic constants, as determined on rock samples from the measurement positions.

The vertical stress results of P1a differed significantly from the elastic model, probably as a result of fracturing of the pillar edge. The reason for the deviation from elasticity will be discussed in greater detail in Section 5.6. Even though the correlation between the vertical stress and the elastic model was not good, the horizontal stress change was similar to the elastic model (Figure 4-52). This finding again suggests that there was very little effect of the foundation fracturing in the dip direction.

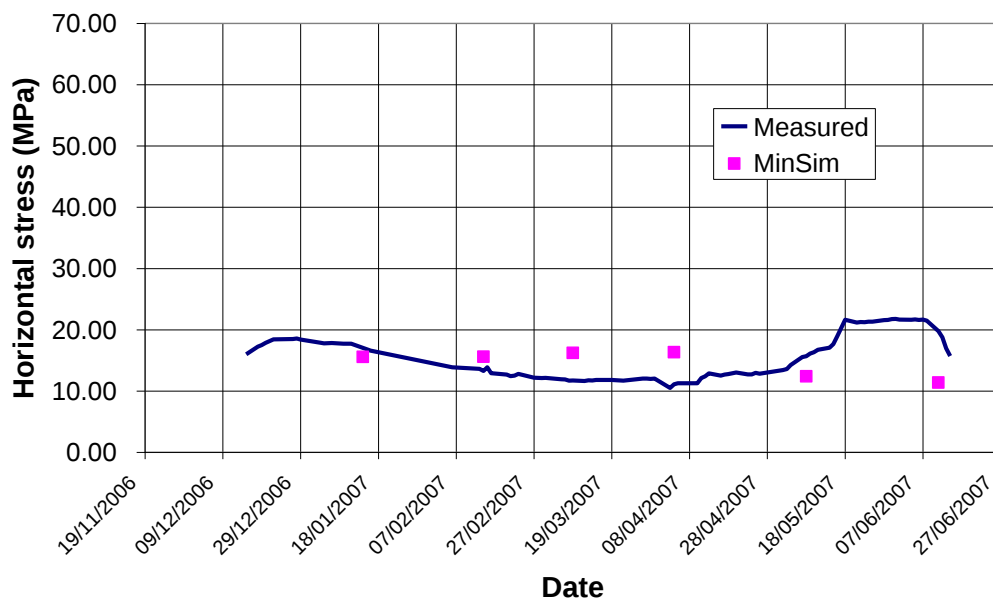


Figure 4-52 Union Site 1: horizontal stress 3.4 m above the edge of the abutment/pillar (P1a)

The vertical stress measurements from P1b were compared to an elastic model and the results are shown in Figure 4-53. The peak measurement was slightly higher but, generally, the measurements compared well with the elastic model.

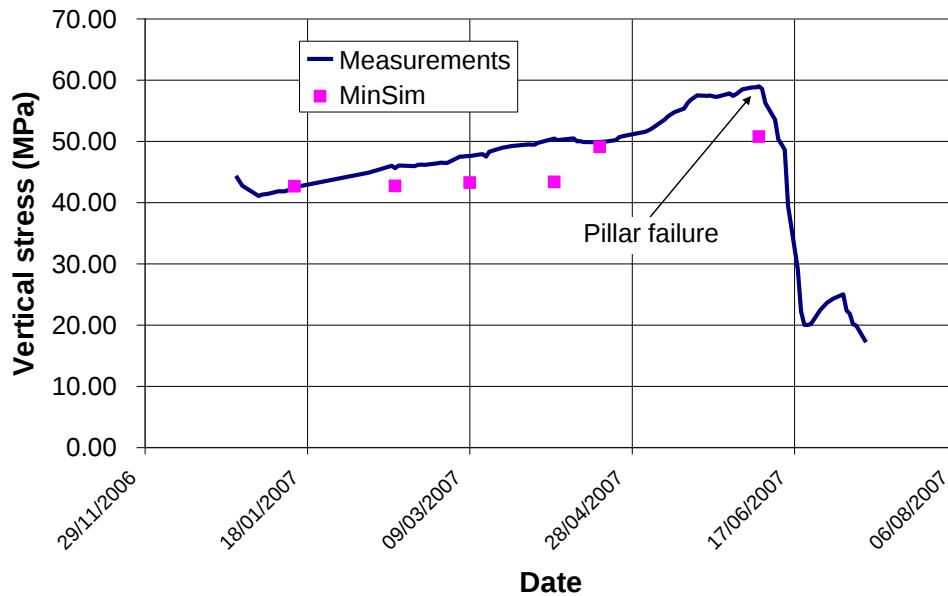


Figure 4-53 Union Site 1: vertical stress change measurements at 5.3 m above the centre of the instrumented pillar (P1b)

Figure 4-53 suggests that the rock mass above the pillar behaved in an approximately elastic manner in the vertical direction. However, in the horizontal direction (approximately on strike) the measured stress deviated significantly from the elastic model from 4 April 2007 (Figure 4-54). This was the date at which mining initiated on the up-dip side of the instrumented pillar. Once again, an increase in compressive stress was measured when the model predicted a decline in this stress. The measured horizontal stress continued to rise even after pillar failure, indicating that horizontal compressive stress was being generated in the strike direction prior to and during pillar failure. The reason for this deviation from the elastic model appears again to be related to foundation damage and the development and dilation of discontinuities and fractures above the pillar.

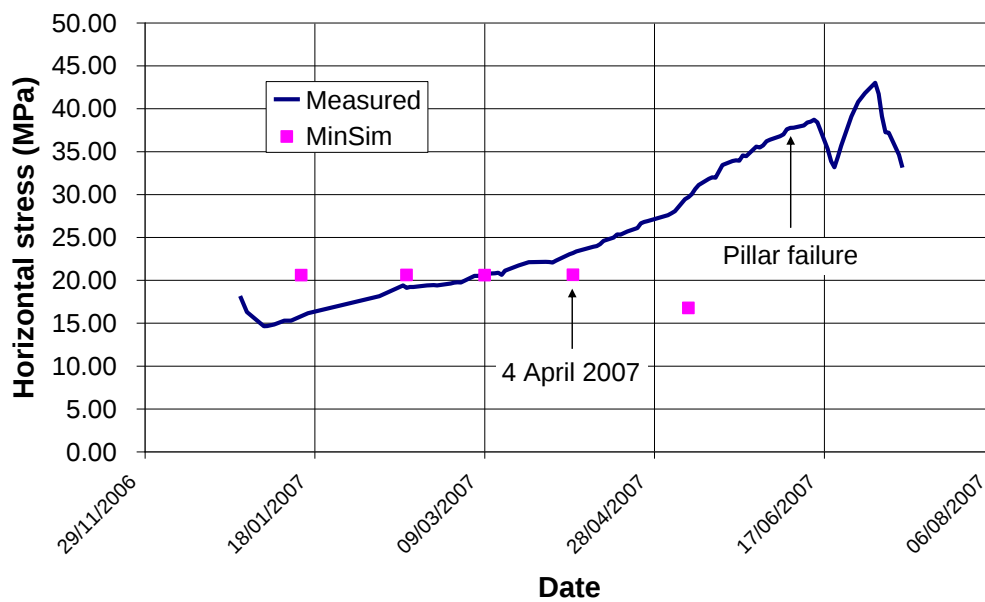


Figure 4-54 Union Site 1: horizontal stress 5.3 m above the centre of the instrumented pillar (P1b)

After pillar failure the instruments were vandalised, which effectively ended the stress monitoring in these two boreholes. However, at a later stage of mining, when the 3s panel was almost mined out, a 3D cell was installed at a height of 6.86 m above the pillar, on its south side (P1c in Figure 4-49). At this position, the field measurements showed that the horizontal stress had dropped to very low levels and was similar to the vertical stress at the date of installation, some three months after pillar failure (Figure 4-55). Subsequently, almost equivalent changes in stress were measured in the vertical and horizontal directions. The results in the figure show the measurements at a point located towards the south side of the pillar – i.e. not over the centre of the pillar. At this location the horizontal stress resulting from fracture development and dilation should have been noticeable. However, the APS was low at the time of installing the instrument and the fracturing of the hangingwall had probably stopped at this stage. The reason for the subsequent drop in horizontal stress is probably due to creep processes that may have been resident in the highly jointed and fractured rock mass.

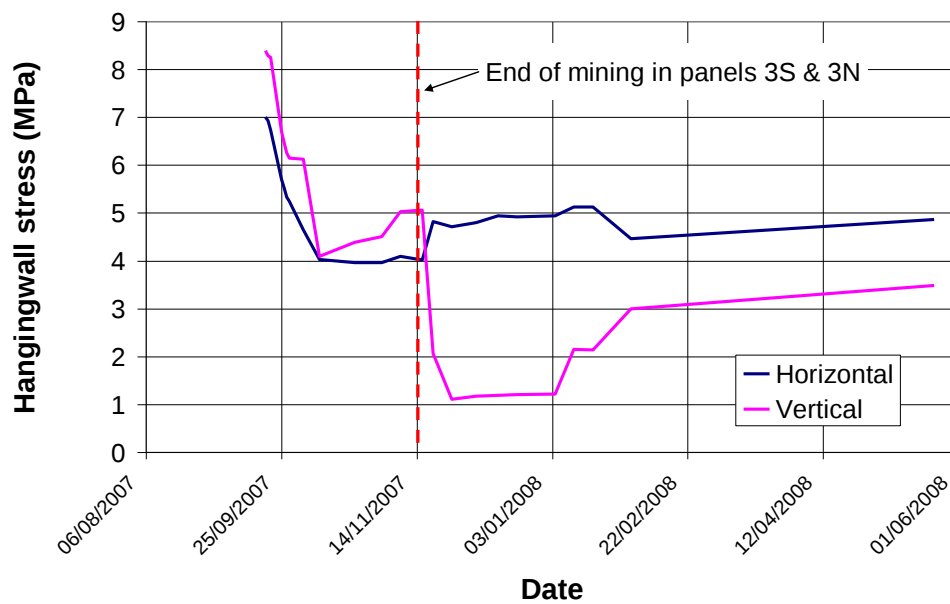


Figure 4-55 Union Site 1: horizontal and vertical stress change 6.86 m above the instrumented pillar (P1c). The horizontal measurement is perpendicular to the pillar edge

A series of horizontal stress measurements was conducted in a vertical hangingwall borehole drilled at the centre of the second stope (Site 2 in Figure 4-56) after mining had been completed. The results of the investigation are plotted together with the borehole camera observations and the elastic MinSim profiles in Figure 4-57. K-ratios of 0.5 and 2.0 were assumed in the plotted model results. (Note the very high levels of horizontal stress in the leuconorites and anorthosites and the comparatively low stresses in the darker norites and pyroxenites.) As borehole breakout was not observed in the borehole camera survey, the quoted stresses in the leuconorites and anorthosites appear to be too high. However, the normal evaluation method was used for these measurements. Also the high stresses coincided with the zone of competent rock observed in the extensometer borehole core. Even if the exact magnitude of these stresses is incorrect, the trend of comparatively higher stresses in the anorthosites is probably true. Interestingly, the laboratory tests from this site showed little nonlinear behaviour even though the virgin horizontal stress was apparently high.

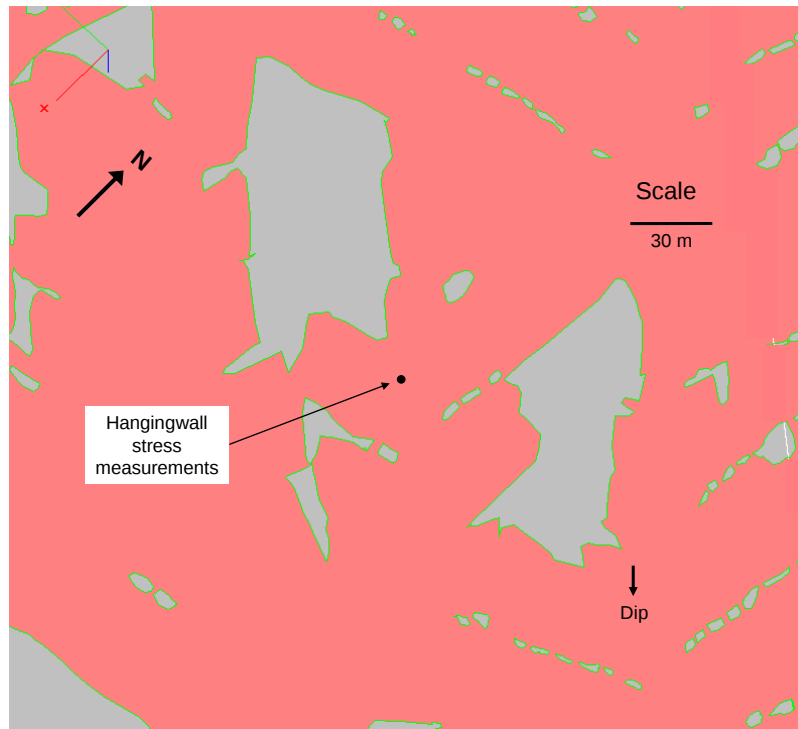


Figure 4-56 Union Site 2: stope sheet showing the location of the horizontal stress measurements conducted in a vertical borehole drilled up into the hangingwall

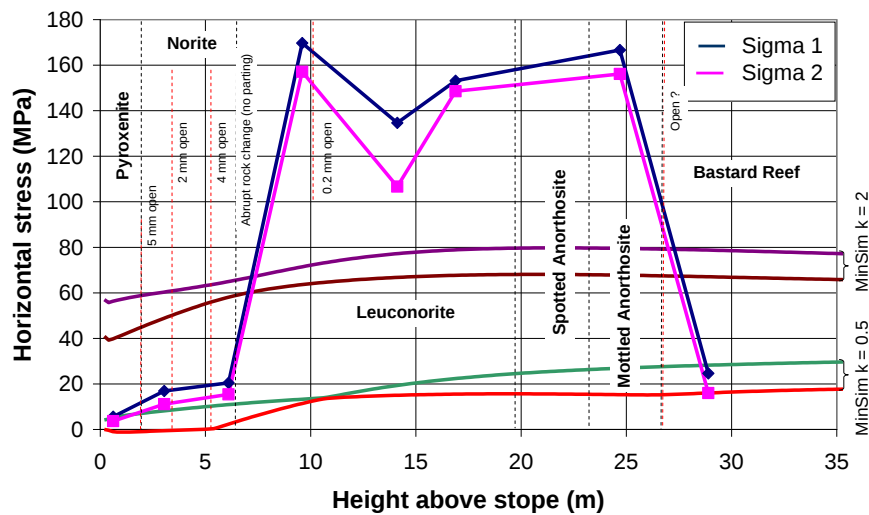


Figure 4-57 Union Site 2: horizontal stresses above the centre of the stope. Sigma 1 and 2 were measured and compared to the elastic results for k -ratios of 0.5 and 2. Red dashed lines show positions of open shallow-dipping discontinuities. Black dashed lines show positions of lithology changes

The measurements in the darker rocks, in the immediate stope hangingwall and the Bastard Reef at the top of the borehole, coincided with the stresses predicted by an elastic model with a k-ratio of 0.5. A similar k-ratio was observed in the pyroxenitic rocks at the Impala site and the trend of comparatively higher stresses in the anorthositic rocks was also apparent at this site. The measurement in the Bastard Reef at the Impala site, however, could have been affected by the adjacent anorthosites, as they were located just below the measurement position. However, the measurement taken at the Union site was about 2 m above the anorthosites and is unlikely to have been affected by the rock change. The measurements suggest the possibility of stress 'channelling', with the anorthositic rocks being in a higher horizontal virgin stress state than the darker, pyroxenitic rocks. These stresses may be tectonic and could explain why horizontal fracturing sometimes occurs in the anorthosites at some distance above the stope hangingwall (Watson et al, 2009).

The open shallow-dipping discontinuities observed during the borehole camera survey (red dashed lines in Figure 4-57) often coincided with changes in lithology (black dashed lines in Figure 4-57). Unfortunately, the horizontal stress was not always measured at appropriate positions, but the formation of beams in the first 10 m is apparent.

The parting just above 10 m (Figure 4-57) occurred well above the theoretical vertical tensile zone of about 2 m and did not coincide with a change of rock type. Even with the effects of the partings and rock-type changes, this tensile zone could not have reached to 10 m above the stope, as shown by the 2D Elfen modelling in Appendix D and by Watson et al (2007c). The fracture was, however, located in the zone of high horizontal stress, and it is highly probable that it formed in extension under high horizontal stresses when the vertical confinement was relieved during mining. The small aperture and the orientation of this plane seem to confirm the extension mechanism.

A detailed list of the fractures and discontinuities observed in the borehole camera survey are provided in Appendix E. Open vertical discontinuities were more prevalent in the low horizontal-stress environment of the pyroxenites, as expected. However, some were also observed up to 7 m above the stope, where

even a k-ratio of 0.5 in the elastic model suggests the presence of a significant compressive stress in the horizontal direction. These observations suggest movement of hangingwall blocks up to a height of about 7 m above the stope.

The hangingwall conditions at this site could be described as blocky. The Schmidt, lower-hemisphere, equal-area stereo net, provided in Section 2.3, suggests the formation of keyblocks. The propensity for hangingwall unravelling was obviously high and keyblocks had to be properly supported. The shallow-dipping fractures that developed in the pyroxenite and norite lithologies may well have occurred on weak planes formed through igneous intrusion/extrusion or metamorphic flow processes, as suggested by the literature.

4.3.4 Discussion

The literature suggests that the formation of beams is common in the Bushveld Complex and that beam formation often occurs through excavation-parallel stress fracturing of massive ground (Swart et al, 2000). Beam formation was observed in the immediate hangingwall of the Amandelbult and Union sites. The top and bottom of the beams generally coincided with a change of rock type, even though these changes are gradational. In some instances at the Amandelbult site, stress profiles consistent with beam formation were measured but no partings were observed in the borehole. These profiles may have resulted from the interactions of different material types.

Voussoir beam formation (as described by Ryder and Jager, 2002) was not observed at the Amandelbult site because of the relatively high virgin k-ratio. As no partings were observed in the hangingwall at the Impala site, no Voussoir beams formed here either. However, at the Union site, there is a possibility that such a beam formed below a parting at about 1.7 m above the stope. The single stress measurement in this “beam” does not confirm a Voussoir beam as the magnitude can also be explained by an elastic model with a low k-ratio in the pyroxenites.

An elastic model with a k-ratio of about 0.5 matched the measured stress profiles in the pyroxenites at both the Impala and Union sites. A k-ratio of 0.5 was also

suggested by the dimensions of sockets (Watson et al, 2005a) in the footwall pyroxenites at the Union site. By comparison, the horizontal stresses in the anorthositic rocks at these two sites appear to have been higher than in the pyroxenitic lithologies. The findings suggest that the original horizontal stresses of the Bushveld Complex, at formation, may have relaxed more in the comparatively more ductile pyroxenites than in the anorthositic rock types. However, the measurements at the Amandelbult site show that this is not a universal law. More research should be done to determine the reason for the different trends at the Amandelbult site.

The contrast between the pyroxenite and anorthosite stresses at both the Union and Impala sites suggests some form of stress 'channelling' in the anorthosites. This 'channelling' may be an explanation for the high degree of fracturing often observed in the leuconorites and anorthosites. The parting observed at 10 m above the stope at Union Site 2 is probably an example of an extension fracture. This fracture appears to have developed when the vertical confinement was relieved during stoping operations. The plane was located well above the vertical tensile zone, even if the influence of the open fractures and parting planes is considered. It appears then that the fracture did not form the upper surface of a beam at this site. Such fractures have been observed elsewhere and may develop high above the panel as described in Section 4.1. Dangerous conditions have been reported where the immediate hangingwall is anorthositic. Such discontinuities may develop from an abutment or face and curve up into the hangingwall, following the stress trajectories.

A comparison between the relative heights of open discontinuities at the three sites suggests that blocky ground conditions are more susceptible to the development of open partings at relatively great heights above the stopes. Elfen modelling (Watson et al, 2007c) showed that the vertical tensile zone is modified when parting takes place on discontinuities, and could be much higher than the theoretical height. However, even this modelling could not explain the heights of the open discontinuities at the Union site, particularly the vertical discontinuities. Similar conditions have also been reported elsewhere (Section 4.1.1). In this case, it appears that the gravitational effects are probably assisted by small-scale shearing on adversely oriented joints when the face is advanced below. This

deformation could result in a complex stress interaction, causing squeezing and opening of vertical and horizontal discontinuities at greater heights above a stope than expected. At Union Site 1, open steeply-dipping discontinuities were observed much higher than the theoretical horizontal tensile zone in an initial survey conducted in the 12 m span. The height of these open discontinuities did not increase when the abutment was advanced away. Since the span increased, the height of open discontinuities should also have increased, unless the abutment was causing shearing on adversely oriented discontinuities. Further research is required to properly understand the behaviour of a blocky hangingwall condition. This mechanism is different to stress 'channelling', where fractures may also develop above the vertical tensile zone under extension.

Almost all of the vertical stress change measurements conducted over the pillars compared well to elastic models when the 'matrix' elastic constants were applied to the evaluations (Section 3.6.5). There were also good correlations between the extensometers and elastic models when these constants were used in the models. The only exception was the set of stress measurements conducted over a pillar at the Union site. However, this result was only slightly higher than the model and may have been influenced by the blocky conditions. All that the instrumentation suggests, therefore, is that where the discontinuities are clamped and micro fractures have not formed, the rock mass behaves in a linear elastic manner, with the same 'matrix' elastic constants as determined on an intact laboratory sample (Section 3.6.5). The joint filling was generally thin at the measurement sites and a thicker filling might have altered the rock mass modulus. The extensometer at the Union site showed that once fracturing or opening of geological discontinuities occurred, the effective modulus of the rock mass dropped as expected.

In every case where horizontal stresses were measured above a pillar, the stress perpendicular to the long axis of the pillars or chain of pillars was always higher than predicted by a MinSim elastic model. Even where the MinSim models showed a decrease in horizontal stress just after pillar failure, the measurements showed a stress increase. 2D FLAC modelling showed that the development of vertical fractures above a pillar can cause an increase in horizontal stress in the hangingwall. Beam development may also have contributed to the horizontal

stress induction in some areas. The induced stress in both instances would be perpendicular to the long axis or line of pillars.

The extensometer installed about 0.9 m above Pillar 1 at Impala showed dilation during pillar failure. At the same time compressional strains were measured over Panel 8s, suggesting an increase in horizontal stress of between 2 MPa and 11 MPa. This stress was estimated from an average strain change over half the dip span of the panel. In reality, the horizontal stress at the pillar edge may have been affected more than at the panel centre.

The horizontal stress above the pillar at the Union site continued to increase for some time after pillar failure, suggesting the continued development or opening of fractures after pillar failure. However, ultimately, the horizontal stress at this site dropped to about the same level as the vertical stress. This final horizontal stress state suggests creep behaviour, probably due to slip on unfavourably oriented discontinuities in the blocky ground. The final horizontal stress condition suggests that the fractured rock above a pillar may behave in an approximately hydrostatic manner, particularly in blocky ground conditions.

Steeply dipping fractures were observed above the pillars at the Amandelbult and Impala sites after pillar failure, where the fractured edges of the pillars had fallen away. In addition, sub-vertical fractures were observed at a height of about 1.2 m above the pillars during drilling operations at the Impala site. These discontinuities were not observed in other similar boreholes drilled 1.5 m above the pillars. However, evidence of fracturing at significantly higher levels was shown by water dripping from discontinuities adjacent to the highly stressed stability pillar at the Amandelbult site.

4.4 Footwall and closure investigations

This section deals with the underground measurements of footwall deformations in the three instrumentation sites. The results of footwall extensometers and stope closures (convergence) are compared to numerical models and observations. A detailed description of the instrumentation and observations is provided in Appendix E.

4.4.1 Shallow-depth good rock mass conditions (Amandelbult site)

Figure 4-58 shows a plan view of the instrumentation site and the location of the closure meters relative to the instrumented pillars. Closure stations were installed on the up-dip and down-dip sides of Pillar 2 (P2) at about 2 m ahead and 3 m behind the down-dip face respectively. The grey colour in the figure represents pillars and unmined areas, while the excavation shape prior to instrumentation installation is shown in red and the array of colours shows the face advances during the monitored period. The site was located at a depth of 600 m below surface.

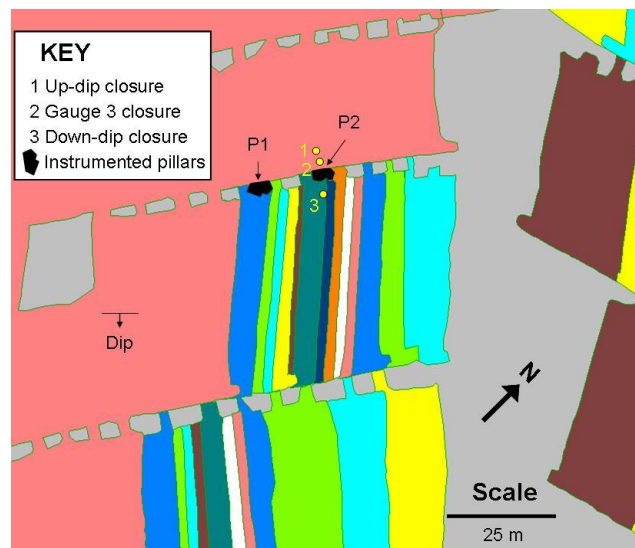


Figure 4-58 Amandelbult site: stope sheet showing the positions of the closure meters around P2

The closure measurements in Figure 4-59 are compared to an elastic MinSim model that was constructed with elastic constants which were determined in the test laboratory, i.e. 80 GPa and 0.23 for the Young's modulus and Poisson's ratio respectively. A pillar residual strength of 20 MPa was assumed in the model, which agrees with the pillar stress measurements discussed in Section 5.6.

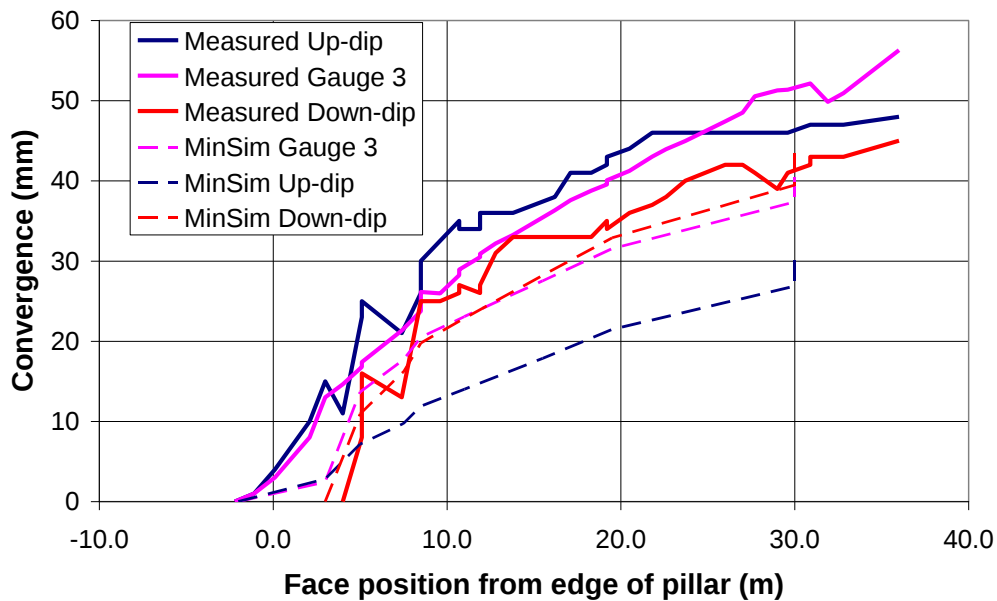


Figure 4-59 Amandelbult site: measured closure and modelled elastic convergence. Up-dip and Gauge 3 on the gully side of pillar

The comparison between the measurements and the elastic model as shown in Figure 4-59 suggests that inelastic closure occurred during the initial face advances on the up-dip side of the pillar. Little, if any, inelastic behaviour was measured on the down-dip side. The subsequent recovery of some of the initial inelastic deformations on the up-dip side, particularly at the Gauge 3 position, suggests that at least some of this closure could be attributed to the inability of the model to simulate pillar/abutment edge failure. The comparatively steeper slope of the measurements in the latter part of the Gauge 3 curve, though, suggests that inelastic closure was occurring around this station.

A borehole camera survey was conducted in a vertical borehole to a height of 15 m above the centre of the stope. Only a few open discontinuities were observed in this survey and the combined aperture of these discontinuities

accounted for only a small percentage of the measured inelastic closure. For this reason it is assumed that most of the inelastic closure was due to fracturing or shearing on existing discontinuities in the footwall. At another instrumentation site at Amandelbult (Watson and Noble, 1997), extensometers at the centre of a panel showed approximately 6 mm inelastic deformation in the footwall and elastic conditions in the hangingwall.

From the closure measurements and the hangingwall observations, it appears that relatively more fracturing occurred below the higher pillar sidewall adjacent to the ASG than on the down-dip side of the pillar. The difference in closure on the two sides of the pillar suggests that the ASG may have had an influence on the fracturing/damage and possibly the positions and orientations of the fractures. Fractures may also have developed prior to pillar formation because the up-dip panel was cut ahead of the down-dip panel and subsequent damage may have been restricted to movement on these existing planes. In addition, the ASG side of the pillar formed a higher sidewall with a resultant lower confinement on that side of the pillar, permitting fractures to form more easily on that side.

The lower rods in both P1 and P2 rotated upwards (Figure 4-60), indicating that the pillars punched into the footwall as described in Appendix E. No rotations were observed on the upper rods on either of the two pillars. The almost-elastic closure measured on the down-dip side of the pillars (Figure 4-59) suggests that most of the fracturing was occurring on the gully side of the pillar (Figure 4-61). The influence of an ASG on fracture development has been observed elsewhere on the Merensky Reef, where a re-raise was developed through an abutment and adjacent 2 m-wide siding between the original abutment and the ASG (Figure 4-62). The figure also shows that the fractures that developed in the anorthosite terminated on the chromitite layer just below the pyroxenite.



Figure 4-60 Amandelbult site: rotation of jumpers grouted into the centre of P2 near the top edge of the pillar and about 1 m vertically below. A = pillar failure; B = three months after pillar failure

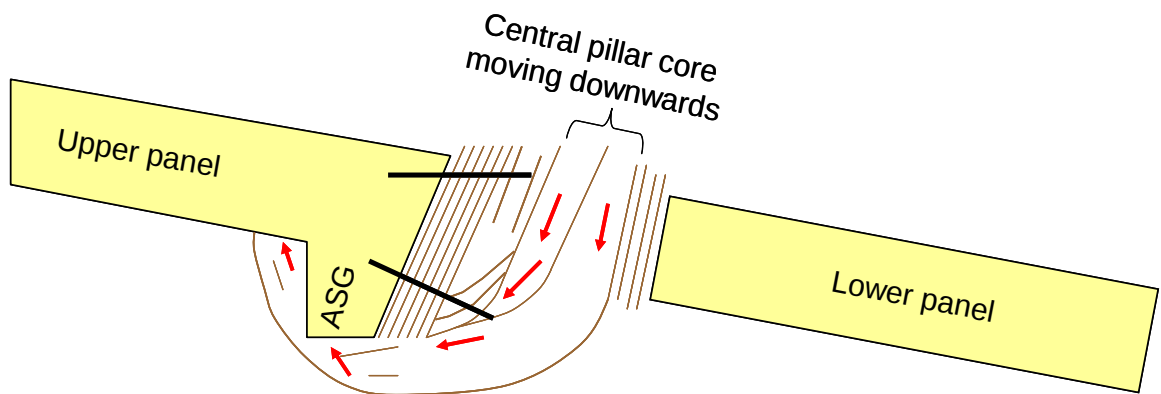


Figure 4-61 Amandelbult site: diagram showing a section through P2, indicating material flow along a simplified matrix of fractures in the pillar and foundation during loading that could explain the rotation of the lower jumper (not to scale)

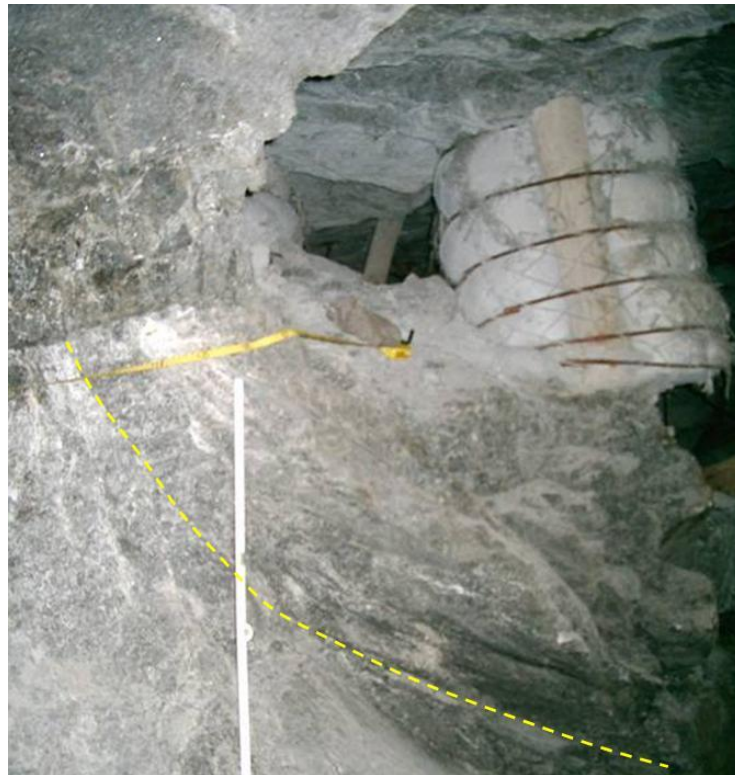


Figure 4-62 Fracturing from the edge of an abutment, curving towards the bottom of the ASG to the right of the photograph. Exposed in a re-raise

The inelastic deformations on the up-dip side of the pillar were particularly noticeable in the early stages of pillar development and failure, to a face advance of about 3 m – i.e. a 5 m face advance from the date of installing the closure stations. The reason for the significant difference between the measured and elastic closure during pillar development and failure is probably because of the growth of fractures in the foundation and the inability of the model to capture edge failure, as described above.

As Station 2 (Figure 4-58) was anchored 0.5 m below the ASG and the bottom of the gully was already 1 m below the stope, it is concluded that inelastic processes shown at this station (Gauge 3 in Figure 4-59) occurred deeper than 1.5 m below the stope. Unfortunately, footwall borehole cores were not available and borehole camera surveys were also not conducted in the footwall at this site.

Figure 4-62 shows that significantly less fracturing occurs in the pyroxenite than the anorthosite rock types for a given stress condition. This difference in

fracturing was also observed at the Impala site where horizontal boreholes were drilled vertically above each other in an abutment. These boreholes were spaced 0.5 m apart, one in the anorthosite and the other in the pyroxenite rock types, respectively. Being in the same abutment the two boreholes were subjected to the same vertical stress. Borehole breakout could be clearly seen in the anorthosite but not in the pyroxenite holes, even though the anorthosites are typically stronger than the pyroxenites. The observations suggest that either fracture development is comparatively easier in the more brittle anorthosites or that there was a higher horizontal stress in the pyroxenites that acted to reduce the stress concentrations on the borehole sidewall. The latter scenario is unlikely, however, as there was evidence of comparatively lower horizontal stress conditions in the pyroxenites elsewhere in the stope, e.g. discing was observed in the anorthosites but not in the pyroxenites when a vertical borehole was drilled from the haulage ahead of the advancing face at the same site.

In summary, there are several possible reasons for the preferential damage of the footwall at this site:

- comparatively higher horizontal stress in the anorthositic footwall;
- comparatively higher brittleness of the immediate footwall;
- effects of the ASG; and
- distribution and orientation of geological structures, e.g. joints and faults.

A likely explanation for the preferred footwall damage at the Amandelbult site is the effect of comparative brittleness. During pillar failure, less tension would have been induced in the pyroxenite hangingwall than the anorthosite footwall as a result of post-failure pillar dilation (pillar and hangingwall are pyroxenite). The preferential fracturing would have assisted the pillar to punch into the footwall. In addition, the geological structures in the footwall were not examined and there may have been some unfavourably oriented discontinuities that could have assisted in the footwall damage. Such structures were identified elsewhere at Amandelbult during a separate instrumentation programme (York et al, 1998). However, the observed stress regeneration is unlikely in the presence of such discontinuities and FLAC models suggest this regeneration only occurs where minimal foundation damage has occurred.

4.4.2 Intermediate-depth good rock mass conditions (Impala site)

A stope-sheet showing the Impala instrumentation site is provided in Figure 4-63. The workings were conducted at a depth of 1100 m below surface. The grey areas in the figure represent unmined areas and pillars, and the array of colours shows the monthly face advances. A line of closure stations was installed down the dip span of Panel 7s as indicated in the figure. The sequence of mining after this installation is numbered in the figure.

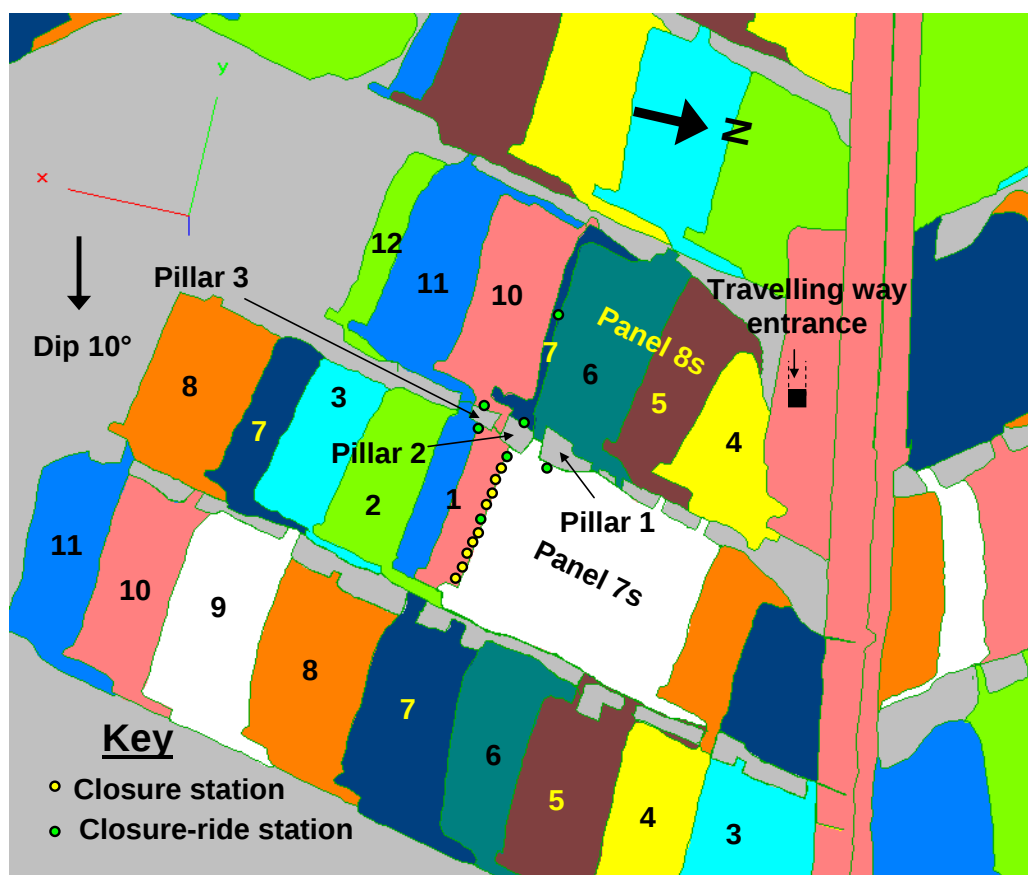


Figure 4-63 Impala site: stope sheet showing the locations of the closure and closure-ride stations and the sequence of mining

As shown by the mining sequence in Figure 4-63, Panel 7s was initially mined under isolated conditions. The closure measured between the face advances of 6.5 m and 32.7 m from the line of closure stations is compared to an elastic model in Figure 4-64. The comparison again shows the inability of the model to simulate the abutment-edge failure. A clearer picture of the early inelastic

deformations is shown in Figure 4-65, where the elastic MinSim convergence was subtracted from the measurements. The constants used in the elastic model were much lower than normal for this rock type because of the micro fracturing that occurred when the panels were mined (Section 3.7). The laboratory tests showed that reasonable elastic constants for the virgin stress conditions of about 30 MPa are 15 GPa and 0.32 for the Young's modulus and Poisson's ratio respectively. The closure measurements are described in greater detail in Appendix E.

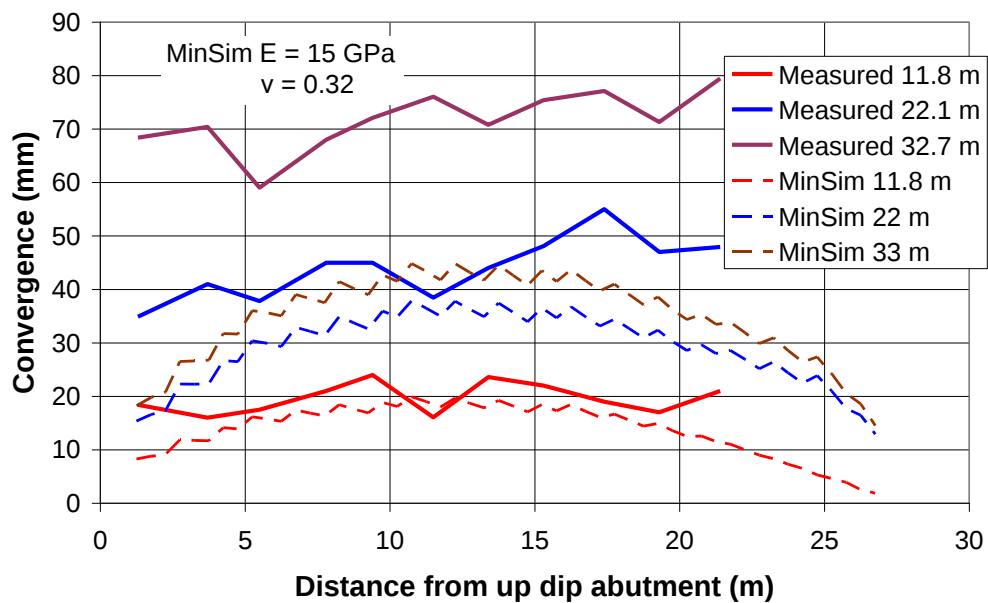


Figure 4-64 Impala site: comparison between the measured closure and MinSim convergence for the isolated 7s panel. Measurements and model initiate at a face advance of 6.5 m from the instrument line

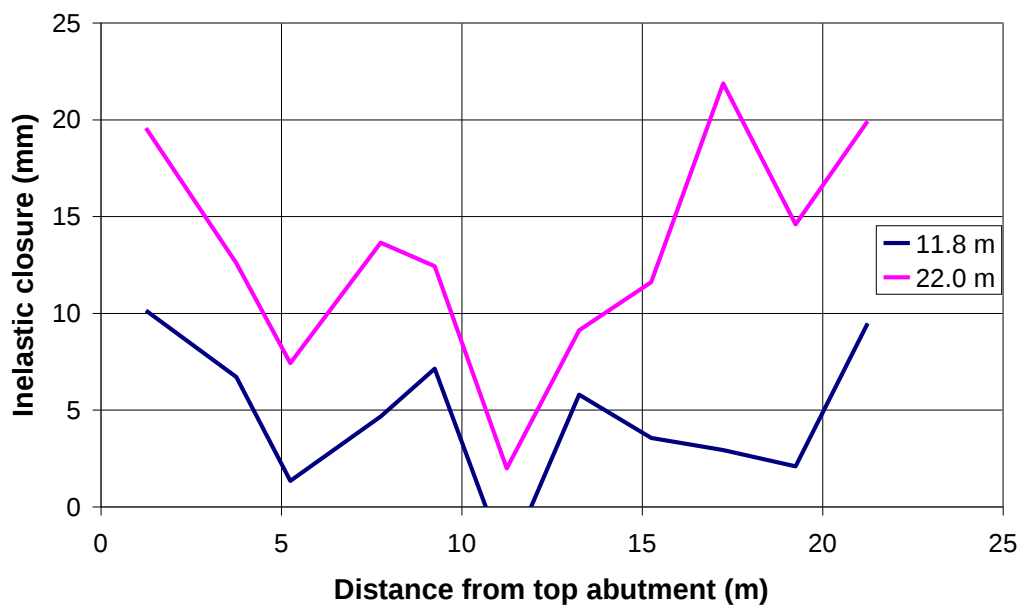


Figure 4-65 Impala site: inelastic closure in Panel 7s during isolated mining conditions. The results show the difference between the closure measurements and the elastic MinSim model ($E = 15 \text{ MPa}$ and $\nu = 0.32$)

Since a constant ‘low’ modulus was assumed in the model and the rock located in vertical stress conditions of about 10 MPa to 20 MPa would not have been micro fractured (Section 3.7.5), the measured closure near the boundaries of the excavation should have been less than indicated by the model. However, a higher closure was measured, suggesting that the inelastic deformations may have been greater than indicated in Figure 4-65. The figure shows that the closure at the centre of the panel was approximately elastic for the first 22 m face advance, (if a modulus of 15 GPa is assumed in the model). FLAC modelling carried out by York et al (1998) suggests that the measured profile of closure (Figure 4-64) is indicative of shallow-depth discontinuities/fracturing and buckling at the abutments. An increase in vertical stress along the abutment, as the face advanced, may also have affected the closure in the vicinity of the abutment.

No fractures were identified in the hangingwall boreholes, except in the boreholes drilled adjacent to and over the pillars (as shown in Section 4.3.2). However, the single footwall hole at about 5 m down-dip of Pillar 2, which was drilled when mining was far advanced in the stope (Figure 4-66), showed many open fractures and discontinuities. The results of this borehole camera survey are shown in

Appendix E. In comparison, relatively few discontinuities were observed in the nearby vertical borehole drilled through the footwall ahead of the 7s and 8s faces (CSIRO1 in Figure 4-66). This difference in the number of discontinuities implies that fractures developed during mining.

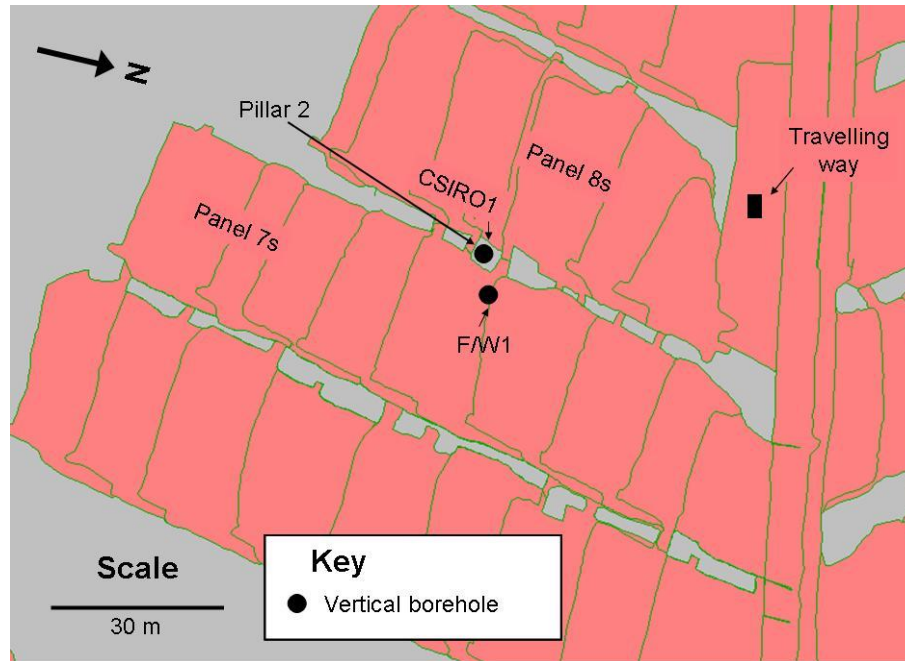


Figure 4-66 Impala site: stope sheet showing the vertical boreholes drilled through the footwall and the location of the travelling way entrance

The closure profiles shown in Figure 4-64 suggested that significant inelastic deformations occurred near the vertical abutments even when the vertical stresses around the face and abutments were low; i.e. little fracturing was observed on the face or abutments (Figure 4-67). The reason for this closure profile is difficult to explain unless an even distribution of fractures overrides the much smaller elastic convergence. This is possible if extension fracturing occurred. A higher closure near the abutments could have occurred due to buckling and shearing of the significantly weakened material. The process of fracturing may have been assisted by the dilating pillars. Buckling was also observed towards the panel centre, indicating that the extension fracturing (spalling) process could also have caused buckling.



Figure 4-67 Impala site: abutment adjacent to the 7s face when this panel was isolated. The abutments near the face were almost unfractured

The strain predicted by an elastic MinSim model (using the 'matrix' elastic constants) at the depth of the observed fracturing in F/W1 (location shown in Figure 4-66) was about 0.24 mm/m. Laboratory tests show a critical strain of 0.2 mm/m for the development of extension fracturing (Watson et al, 2005b), which indicates that an even distribution of fractures might have occurred as a result of extension fracturing. The blast may also have contributed to the footwall damage, as the timing of blastholes was designed to protect the hangingwall.

At least some of the fracturing probably occurred in extension when the vertical confinement was reduced during mining. It is likely, therefore, that the observed footwall fracturing occurred near/at the face. Once the fracturing had occurred, these fractures would have had little effect on subsequent closure near the centre of the panel. Thus, assuming the closure meters were installed after the fracture formation, the inelastic effects of fracture formation would not have been measured. However, further fracturing is likely to have occurred in the weakened footwall near the abutment, which explains why inelastic closure was measured in that region.

The closure-ride station at the centre of the dip line of closure stations in Panel 7s was installed 0.4 m from the face (Figure 4-63). The measured closure at this station was again compared to an elastic MinSim model, with constants of

15 GPa and 0.32 for the Young's modulus and Poisson's ratio, respectively. The closure for the period during the isolated mining conditions is shown in Figure 4-68. Although the station was included in the same line of closure meters described above, it was installed much closer to the face than the other instruments were. This allowed early closure to be monitored at the centre of the panel. The early elastic convergence could not be compared to the measured closure due to the inability of the model to capture the development of the nonlinear behaviour or the observed extension fractures. A relatively high, 'matrix', modulus was probably present over the face area, which only became nonlinear at some distance back from the face. This distance appears to be between 5 m and 8 m in Figure 4-68. The underground measurements suggest that the effects of the relatively high modulus about the face influenced closure for a face advance of about 6 m. Further face advances resulted in an approximately elastic behaviour as shown by the dashed curve in Figure 4-68. It is not clear why the measured closure did not increase to meet the MinSim convergence once the effects of the higher modulus rock had dissipated.

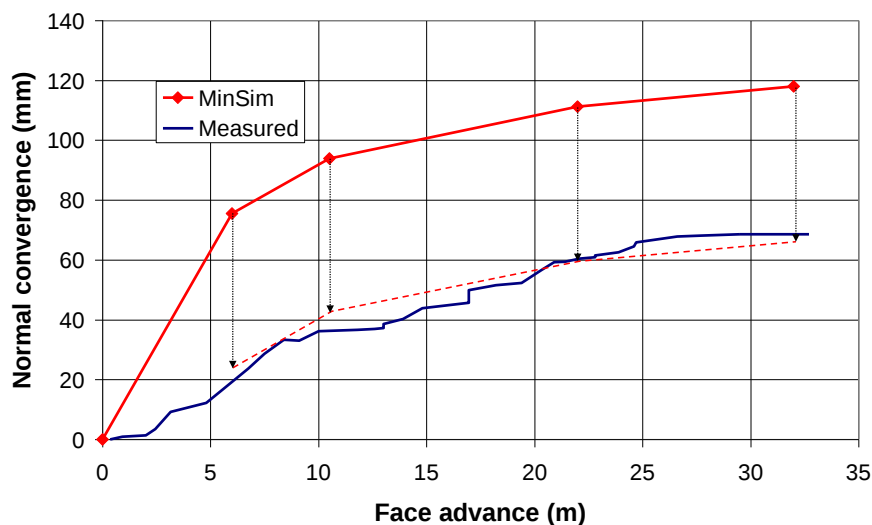


Figure 4-68 Impala site: measured closure and elastic convergence at the centre of the panel. The dashed line compares the modelled convergence curve to the measurements from a face advance of 6 m

The relatively smaller measured closure could also be explained if it is assumed that the face was crushing and some of the elastic closure occurred ahead of the visible face position. However, it is unlikely that significant closure occurred

ahead of the face as such conditions would be accompanied by regular, steeply dipping extension fractures in the hangingwall and footwall. These fractures were not observed. In addition, the vertical sidewalls of the abutments were almost unfractured near the face (Figure 4-67), suggesting a relatively low vertical stress condition.

Lateral movement may have occurred on shallow-dipping discontinuities/fractures in the footwall. Such displacements appear to have occurred on a discontinuity located about 1.3 m below the 8s panel (Figure 4-69). Even relatively small shear displacements would have resulted in extensive open discontinuities. Open planes were apparent during the drilling of the extensometer borehole in the footwall of Panel 7s. At about 2.5 m below the stope there was a sudden loss of water. Water ingress was also observed from fractures/discontinuities at about this depth during a borehole camera survey. Such lateral movements along discontinuity surfaces probably also contributed to the closure. Spencer and York (1999) measured significantly more deformation in the strata above a parting plane some 4 m in the footwall at another instrumentation site at Impala Platinum.



Figure 4-69 Impala site: strata-parallel fault observed in the travelling way, about 1.3 m below the stope

FLAC modelling suggests that a single fault plane with a friction angle of about 30° is unlikely to influence pillar behaviour. In addition, very few geological discontinuities were observed in the footwall. It is therefore unlikely that geological discontinuities had a significant influence on the observed footwall damage at this site.

Very little dip ride was measured adjacent to the pillars, even though significant inelastic closure was measured after these instruments were installed. However,

most of these instruments were installed after pillar failure, which means that some ride could have occurred before installation. The dip ride at the centre of Panel 7s was elastic after pillar failure, suggesting that slip on discontinuities did not occur at that stage. The instrumentation results suggest that there was little dip ride associated with pillar punching after pillar failure.

A vertical footwall extensometer (CSIRO1 in Figure 4-66) showed the deformation that occurred between the bottom of Pillar 2 and the footwall drive, some 14.5 m below. These results are compared to an elastic model in Figure 4-70. Positive deformations in the graph are compressive. The ‘matrix’ elastic constants for the footwall anorthosite (110 GPa and 0.28 for the Young’s modulus and Poisson’s ratio, respectively) were applied in the model. An excellent correlation is shown between this model and the measurements prior to pillar failure, suggesting that the rock mass was behaving in a linear elastic manner with the same modulus as a small, intact rock sample. The under-estimation of the deformations by the model after pillar failure (~2 mm) could be explained in three ways:

- application of the wrong pillar residual strength in the model (30 MPa);
- fracture development; and
- shearing of the borehole.

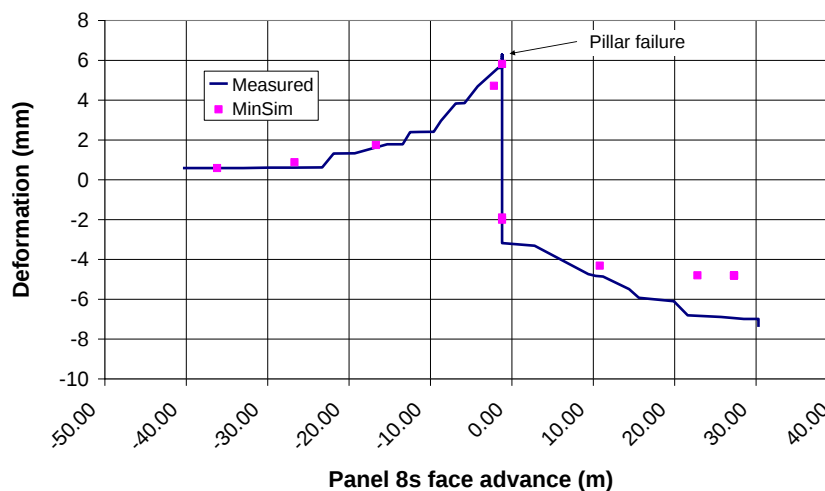


Figure 4-70 Impala site: footwall extensometer below the centre of Pillar 2, plotted against the advance of the 8s face. Negative face advances represent the face position behind the instrument. Positive deformation represents compression

The stress measurements performed above the pillars show that the residual strength of the pillars was about 30 MPa. There was also good agreement between the model and the closure measurements around the time of pillar failure when this residual strength was assumed in the models. Thus it appears that the assumed residual strength was approximately correct.

The development of suitably oriented fractures in the footwall below the anchor point would have increased the extension deformation at the measurement point and, could, therefore be an explanation for the greater-than-elastic deformation. However, the punching effect of the pillar would have had an opposite effect (compression) on the extensometer readings. The two mechanisms of inelastic deformation could have worked against each other and it is impossible to determine the contributions of each in the measured deformations. In addition, a 10 mm-diameter cable was sheared off in the borehole just after pillar failure. This shearing probably resulted in additional, apparent extension deformation. Unfortunately, the amount of shearing cannot be established from the results, but was probably small since no ride was measured from the time the closure-ride stations were installed. The extension deformation would have been less than the elastic model if the measurements included only the effects of the punch. The compressive and tensile strains that occurred above (Figure 4-30) and below (Figure 4-70) the pillar, respectively, suggests that the pillar may have moved upwards into the hangingwall.

The lack of fracturing in the hangingwall over the 7s panel indicates that all of the inelastic deformation was probably associated with footwall fracturing. This was surprising as the extensometer installed through Pillar 2 indicated less pillar punching in the footwall than in the hangingwall at pillar failure. In addition, the anorthosite footwall was slightly stronger than the pyroxenite hangingwall. Observations made in the footwall drive and travelling way (also excavated through the footwall) suggest that the horizontal stresses in the anorthosite footwall may have been higher than the measured stress in the pyroxenite hangingwall. Blasthole socket dimensions (Figure 4-71) suggested a k-ratio of about 1.4 in the footwall drive (Watson et al, 2005a). Core-discing was also observed in the immediate footwall of the reef during the drilling of CSIRO1 (Figure 4-72). The discing suggests that the k-ratio may have been even higher

than 1.4 at this location. However, the stress condition in the borehole may have been exacerbated by the approaching face, which was about 5 m behind the hole at the time of drilling. Since this discing was not observed in the immediate hangingwall pyroxenite, it appears that there was a relatively higher stress in the footwall. A stress measurement suggested a k-ratio of about 0.5 in the immediate hangingwall. The initial hangingwall punching observed in the extensometer may, therefore, have been due to a lower confinement in this lithology. However, Prandtl-type fractures did not form in the hangingwall, possibly because of the comparatively more ductile nature of pyroxenite than anorthosite. Peak pillar strength appears ultimately to have been controlled by the footwall.



Figure 4-71 Impala site: blasthole sockets indicating a k-ratio of about 1.4



Figure 4-72 Impala site: core-discing observed in the F/W anorthosites

The in-stope footwall extensometer in Panel 7s (FW1 in Figure 4-66) was installed when Panel 8S had been stopped and Panels 7S and 6S were very far advanced. The instrument measured deformation between the stope and 12.57 m below the stope. These measurements were made for 12 months after Panels 7S and 6S had reached their final stopping positions. Few or no inelastic deformations were measured from the time of installation (Figure 4-73).

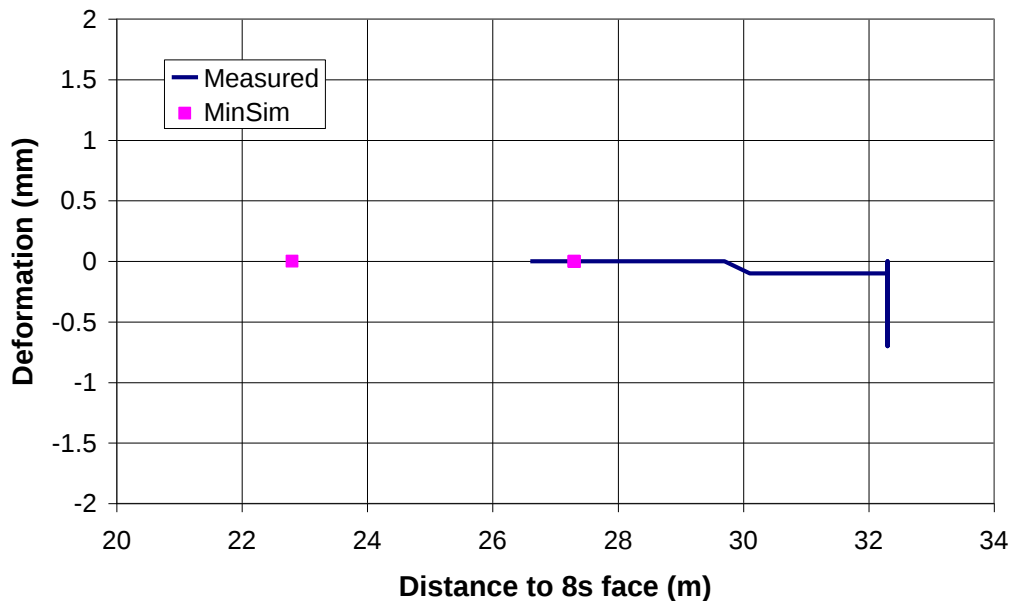


Figure 4-73 Impala site: results of the footwall extensometer installed in F/W1 (Figure 4-66), monitoring deformation between the stope and a depth of 12.57 m below the stope

After pillar failure, a 2 m-deep slot was cut into the footwall adjacent to P1 (Figure 4-74). The fracturing that occurred prior to the slot being cut was thus exposed, and is shown in Figure 4-75. Some of this fracturing probably occurred before the siding was cut. At this stage the pillar was a 5 m-wide strip-pillar. Other fractures may have developed subsequent to the cutting of the siding, during the failure of a comparatively narrow pillar. The observed fractures are similar to the FLAC results described in Section 4.2 and shown in Figure 4-76. The fracturing shows that the foundations are affected during pillar failure and, therefore, that pillar behaviour is influenced by the foundation properties. Both Prandtl wedge- and Hertzian crack-type fractures were observed.



Figure 4-74 Impala site: holing slot cut through the siding from the ASG, after pillar failure. P1 is to the right of the photograph



Figure 4-75 Impala site: fracturing and jointing observed in the slot adjacent to P1. The fracturing was exposed when a holing was cut through the footwall after pillar failure. The yellow line indicates the position of a shallow-dipping, curved fracture. Vertical fractures can also be seen

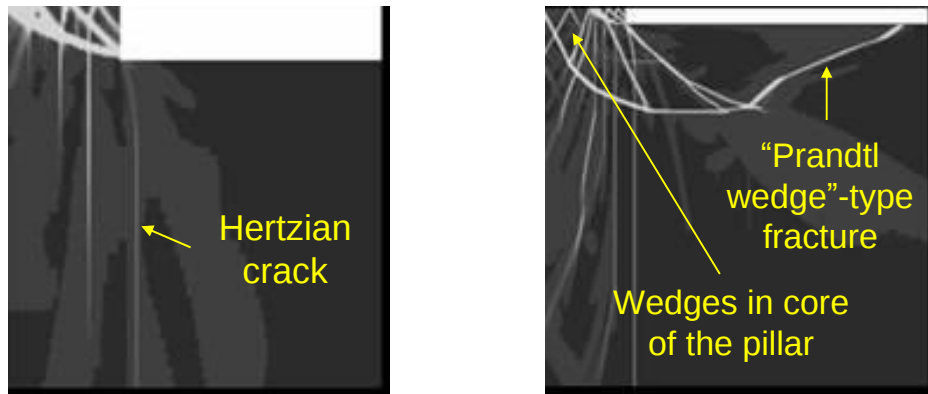


Figure 4-76 *FLAC failure distribution, using dense mesh and ductile material; $w/h = 2.0$ (left) and 5.0 (right) (double symmetry)*

4.4.3 Intermediate depth poor rock mass conditions (Union Spud shaft)

The Union site (Figure 4-77) was located in blocky hangingwall conditions, at a depth of 1400 m below surface. The footwall rock type was unusual and different from that at the other two sites because the reef was located in a large pothole. This means that the Merensky Reef was located in a lower stratigraphical sequence than normal. However, the hangingwall sequences remained the same except for the lithological thicknesses, which were greater.

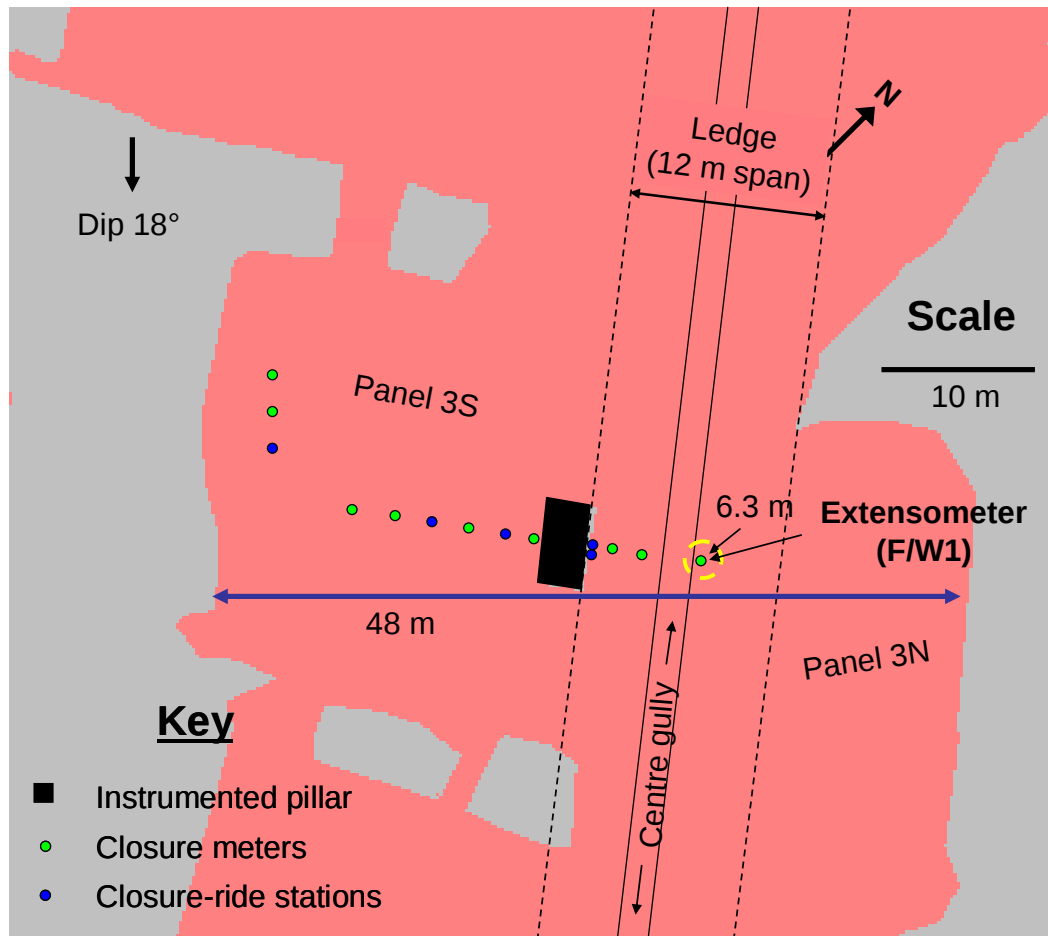


Figure 4-77 Union site: stope sheet showing the locations of the closure and closure-ride stations and the footwall extensometer

Closure stations were installed in the ledge prior to the mining of the panels and closure was monitored for the duration of mining in the stope. Figure 4-78 shows a comparison of the closure measurements and a MinSim model, with elastic constants of 15 GPa and 0.32 for the Young's modulus and Poisson's ratio respectively. Two closure-ride stations were installed adjacent to each other at 0.15 m from the pillar (0.15 ma and 0.15 mb). Stations were also installed at 2.05 m and 3.1 m from the pillar edge. At 6.3 m from the pillar edge closure was measured between the hangingwall and an anchor located at 5.5 m below the stope. The elastic constants used in the model were selected to match the closure measurements at this closure station.

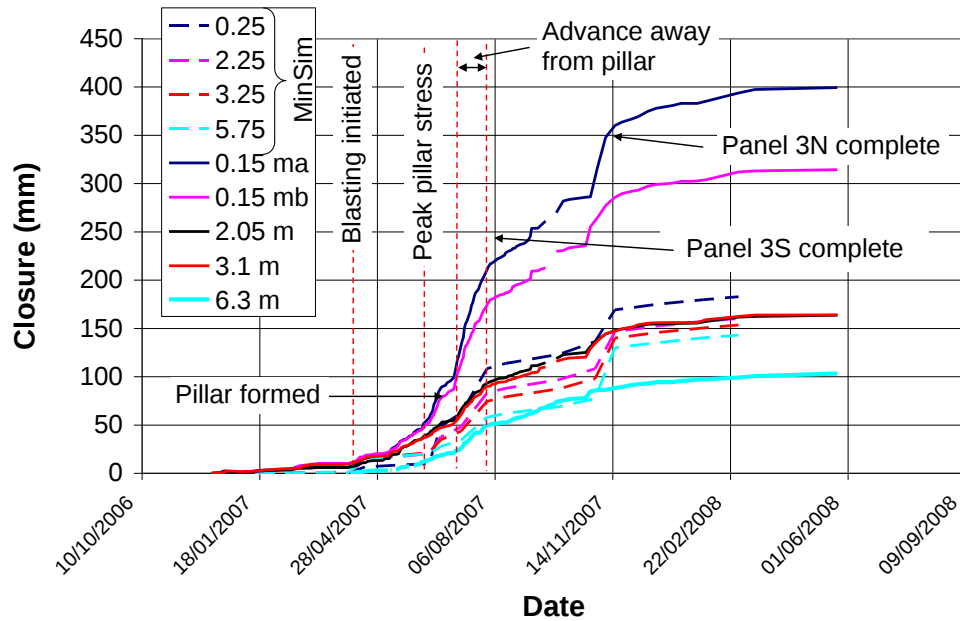


Figure 4-78 Union site: closure measured in the ledge compared to an elastic MinSim model ($E = 15 \text{ GPa}$ and $\nu = 0.32$)

The Young's modulus required in the model to simulate the measured closure was much lower than that determined on laboratory samples and probably accounted for some of the fracturing. A good correlation was observed between the low-modulus model and the closure measured at 6.3 m from the pillar until the mining of Panel 3n initiated. This correlation was not observed on the other closure stations, when the same modulus was used, indicating that significant inelastic closure was probably occurring in the upper 5.5 m of the footwall. This inelastic closure was particularly evident in the two stations at 0.15 m from the pillar edge. These stations also showed variation in deformation along the length of the pillar.

Prior to stoping operations, the closure stations at 0.15 m, 2.05 m and 3.1 m all showed a similar closure of about 8 mm over several months. This observation suggests that the abutment had little effect on the fracturing until mining was initiated on the up-dip side of the pillar.

The closure profiles across the ledge measured during and after pillar formation were compared to a MinSim elastic model with elastic constants of 15 GPa and 0.32 for the Young's modulus and Poisson's ratio, respectively (see Figure 4-79).

The difference in the shape of the measured and modelled profiles shows that significant inelastic movement occurred between 0 m and 2 m from the pillar edge. This was observed even when mining had taken place only on the up-dip side of the pillar. At this stage, little yielding or punching of the abutment was likely but vertical fracturing might have occurred near the edge of the abutment. During pillar formation and failure, there was a sequence of decreasing closure rates with distance from the pillar (Figure 4-79). The profiles in the figure suggest significantly greater fracturing or shearing on fractures/discontinuities near the pillar/abutment and possibly pillar yielding and punching once mining had initiated on the down-dip side of the pillar. A good comparison exists between the FLAC buckling models performed by York et al (1998) and the measurements, particularly where the rigidity of the pillar foundation system is soft in the vicinity of the pillar. The relevant model assumed partings/fractures at shallow depth below the footwall. Additional fractures may also have developed due to the higher stress conditions adjacent to the pillar and Prandtl wedge-type fractures might have developed as a result of pillar dilation. Whatever mechanism was responsible for the high closure adjacent to the pillar, it is clear that the effects of this mechanism were restricted mainly to the immediate vicinity of the pillar.

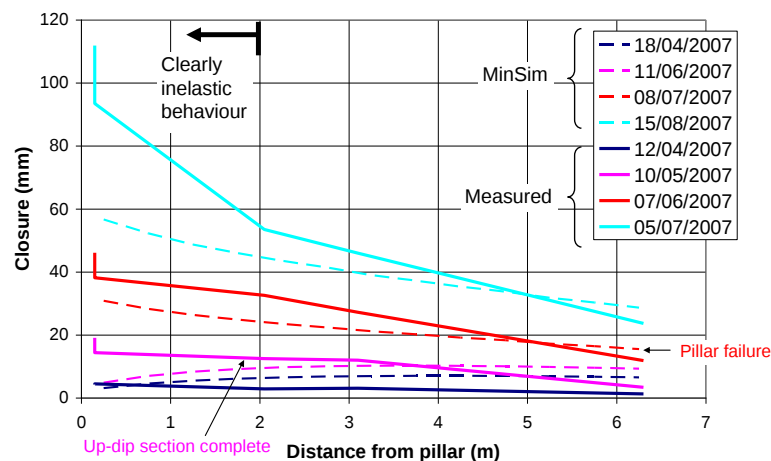


Figure 4-79 Union site: measured closure profile across the ledge (0 m - 6.3 m from the pillar edge) compared to an elastic MinSim model ($E = 15 \text{ GPa}$, $\nu = 0.32$)

The discontinuities and fractures in the footwall were investigated through a vertical footwall borehole (FW1 in Figure 4-77), prior to stoping. At the time, the

strike span was only 12 m, but a high degree of fracturing/jointing was observed down to 3 m below the stope (Figure 4-80). The geotechnical log suggested a relatively weaker footwall than the other two sites and possibly a high horizontal stress in some lithologies (the borehole camera survey results are provided in Appendix E). In particular, the small pieces of core in the Pseudo Merensky between 2.59 m and 3.19 m below the stope (Figure 4-80) may be the result of discing, indicating high horizontal stresses.



Figure 4-80 Union site: core from the extensometer borehole, showing the fracturing/discontinuities in the Pseudo Merensky

The discontinuities observed in the first few metres of this borehole are not observed in other boreholes drilled ahead of mining excavations, suggesting that some of the discontinuities were mining-induced fractures. Since the majority of the discontinuities were parallel to the excavation, it is assumed that these discontinuities are mining induced. At a span of 12 m, an open discontinuity with significant water ingress was observed at a depth of 2.15 m. As the water ingress continued for some hours after flushing the borehole with air, this discontinuity is presumed to be persistent and open over a wide area. Some shearing is likely to have occurred here.

The influence of geological discontinuities on the pillar punching preferentially into the footwall might have been high at this site. However, the fracturing that appears to have occurred prior to pillar formation cannot be attributed to jointing. In addition, the hangingwall was also highly jointed, with steep and shallow-dipping discontinuities.

The formation of fractures is difficult to explain at such small spans and vertical stress conditions, except if extension fracturing occurred. Under the circumstances, such fracturing suggests reasonably high horizontal tectonic stresses in the lithologies where the fracturing took place. The extension

fractures would have formed when the vertical confinement was relieved due to mining. Extension fracturing would also explain the similar closure measured at 2 m and 3 m from the pillar. The required horizontal stress condition was suggested by the apparent discing of the core in the Pseudo Merensky Reef (Figure 4-80).

The extensometer installed in borehole F/W1 was a single anchor instrument measuring deformations that occurred between the bottom of the centre gully (1 m below the stope) and 19 m below the stope. A comparison between the results of F/W1 and a similar hangingwall extensometer (H/W1) shows that more deformation occurred in the footwall (Figure 4-81), particularly in the initial stages of mining. The first sharp increase in the deformation rate coincided with the pillar failure and subsequent mining of Panel 3s. The second sharp increase in closure rate occurred when the 3n panel started mining. At this stage greater deformations occurred in the hangingwall, probably because the abutment that was supporting the hangingwall was advanced away from the extensometer, thus allowing gravity to open up some of the existing fractures and discontinuities.

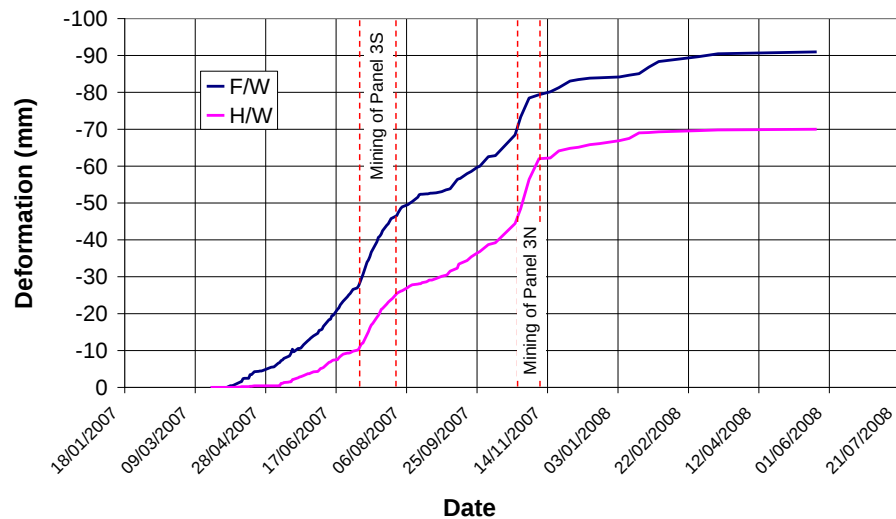


Figure 4-81 Union site: comparison between the footwall and hangingwall extensometer results

The closure meter at 6.3 m from the pillar was anchored 5.5 m below the stope. In order to establish a crude estimate of the component of footwall deformation that occurred above this anchor, the measured deformation was subtracted from

the closure that occurred at 3 m from the pillar. The results, therefore, represent the deformation that occurred between the anchor and the footwall, and are plotted with the footwall extensometer results in Figure 4-82. The region where the two graphs plot on top of each other shows the period where most of the footwall deformation occurred within 5.5 m of the stope, i.e. particularly before pillar failure. After pillar failure there appears to have been more, deeper level inelastic deformation. This might have occurred as a result of either deeper extension fracturing resulting from further relief of the vertical stress or of foundation fracturing as a result of stress transfer to the nearby abutment, approximately 5 m from the extensometer. The latter mechanism is unlikely to have dominated due to the relatively low stress transfer. However, there might have been a complementary effect of the latter on the former mechanisms due to a resultant increase in horizontal stress near the abutment. When the 3n panel was mined, the elastic convergences of the 3 m and 5.5 m closure meters were shown to be significantly different. For this reason the comparison used to evaluate the shallow-depth fracturing was no longer valid and has, therefore, not been plotted in Figure 4-82.

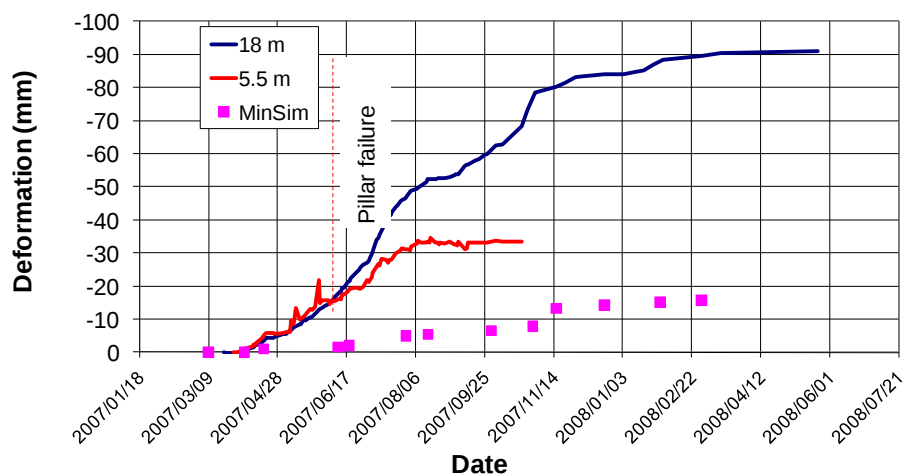


Figure 4-82 Union site: footwall extensometer results showing the deformations that took place down to depths of 5.5 m and 19.0 m below the stope. The anchor at 18 m measured from the bottom of the centre gully; deformations above 5.5 m estimated from modified closure station at 6.3 m from the pillar. The deformations are compared to a MinSim model ($E = 60 \text{ GPa}$ and $\nu = 0.32$)

The results of the extensometers are also compared to modelled closure from a MinSim elastic model. The model was constructed using average elastic constants that were determined in a laboratory, i.e. 60 GPa and 0.32 for the Young's modulus and Poisson's ratio, respectively. The measured deformation could not be replicated by the model, except for the very initial deformations, suggesting inelastic behaviour. The difference between the measurements and the model results suggests that significant inelastic deformations took place.

The two closure-ride stations installed at 0.15 m from the pillar edge showed that relative to the hangingwall, the footwall moved away from the pillar during pillar formation (Figure 4-83). A change in the rate of this ride corresponded to a change in the rate of pillar dilation. This dilation was measured with a horizontal extensometer through the centre of the pillar (Figure 4-84). No elastic strike ride was shown by the MinSim model, indicating that all the measured ride was inelastic. These results suggest that some form of footwall punching or buckling occurred. The closure stations adjacent to the pillar also showed significantly more inelastic closure than the stations at 2 m and 3 m from the pillar (Figure 4-78). This difference in closure suggests that the footwall immediately adjacent to the pillar was affected significantly more than at 2 m from the pillar edge. Fracturing probably also occurred in the footwall at 2 m and 3 m from the pillar edge, though, as a low modulus was required for the MinSim comparison (Figure 4-79). The closure measurements at 2 m and 3 m from the pillar were, however, similar, which suggests that the pillar punching or foundation fracturing/deformation were occurring adjacent to the pillar or at deeper level. It is possible that the fracturing was affected by the centre gully. This centre gully was effectively a slot cut into the footwall at about 4.3 m from the pillar edge (Figure 4-77). Very little footwall ride was measured after stoping had been completed, and there was also a significant reduction in pillar dilation at this time (Figure 4-84).

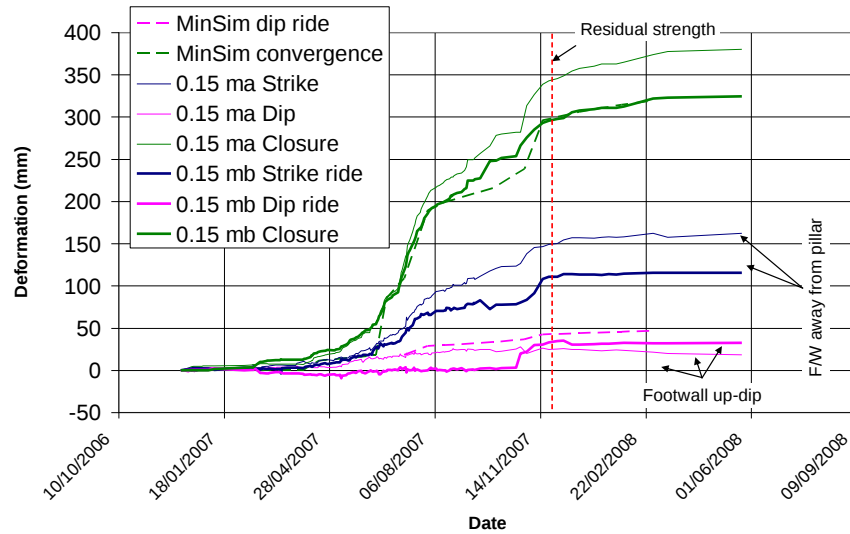


Figure 4-83 Union site: closure and ride measured at station 0.15 m from the pillar edge (0.15 ma and 0.15 mb) compared to an elastic MinSim model ($E = 8.6 \text{ GPa}$, $\nu = 0.32$)

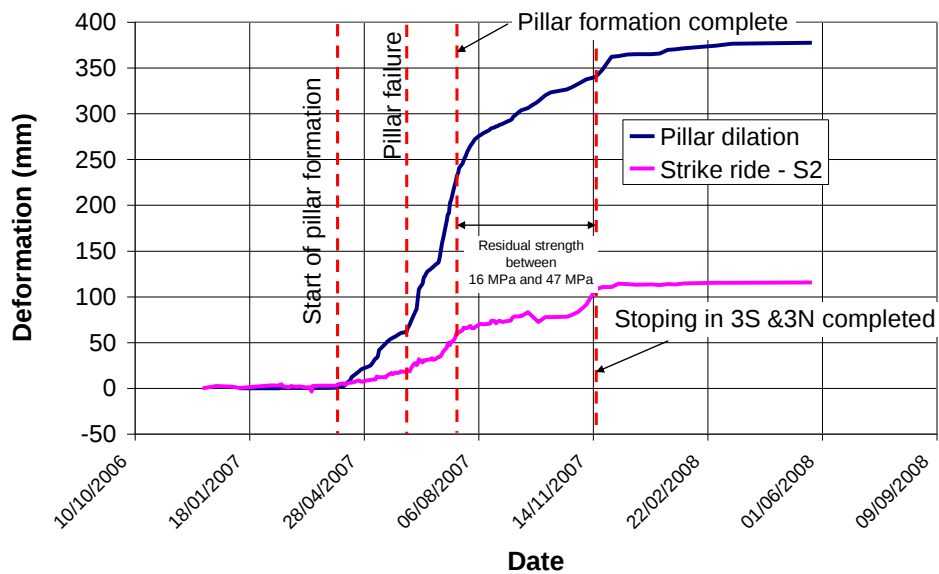


Figure 4-84 Union site: pillar dilation and strike ride at 0.15 m from the pillar, in the ledge

Both the MinSim elastic model and the measurements in Figure 4-83 show a net up-dip footwall ride. A reasonable match between the model and the measurements was achieved for both the closure and dip ride when the Young's

modulus was dropped to 9 GPa in the model. The fractured rock mass at these locations appears to have behaved in an approximately elastic manner, with a much lower modulus during the monitored period.

The closures that occurred within the 3s panel (south of the monitored pillar) are shown together with the results of an elastic MinSim model in Figure 4-85. The MinSim profiles were generated using the same elastic constants as in previous comparisons (15 GPa and 0.32 for the Young's modulus and Poisson's ratio, respectively). The inconsistency in the shape between the modelled and measured profiles shows that inelastic behaviour occurred in the panel. The extent of the fracturing is again indicated by the very low modulus required in the model to simulate the observed closure. Comparisons between the shapes of the elastic and measured profiles suggest foundation damage adjacent to the pillar and near the abutment (19 m from the pillar edge). The inelastic deformation occurred adjacent to the pillar even though the stress levels on the pillar were only between 32 MPa and 47 MPa during the investigation. Additional closure might also have occurred at the face through the influence of a fault, located just ahead of the final face position. The stope was prematurely stopped because of poor ground conditions.

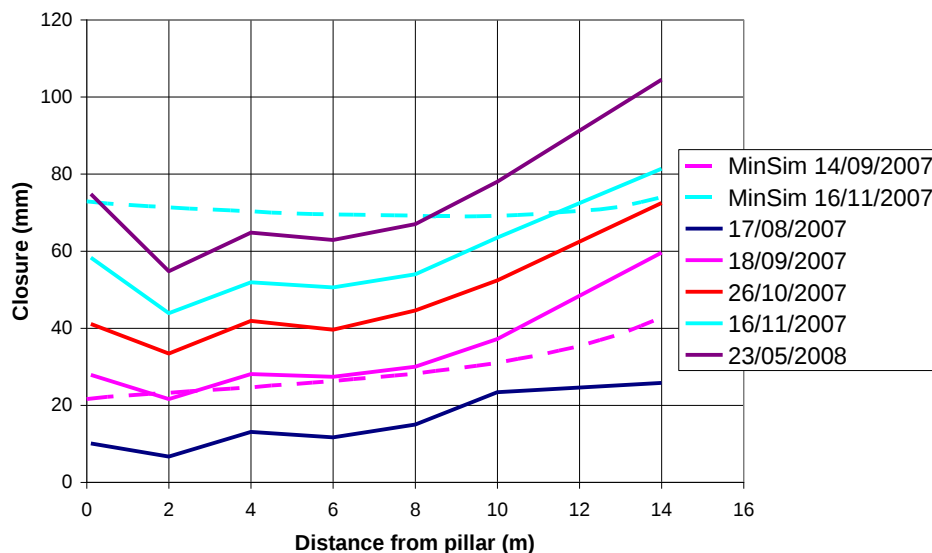


Figure 4-85 Union site: closure profiles across the strike span of Panel 3s from the instrumented pillar to the face at 19 m from the pillar

The inelastic deformations measured adjacent to the pillar and abutment/face in Panel 3s seem to suggest that both the pillar and abutment dilate into the stope, even at fairly low vertical stresses in the case of the pillar. The results are similar to the closure on the north side of the pillar and again suggest footwall buckling or shear along Prandtl-type fractures. The ride measurements, however, showed little movement either in the strike or dip direction during mining. The absence of strike ride at the measurement positions during mining may have been due to opposing movements from the pillar and face. A slight drift of the footwall away from the pillar was detected at 2 m from the pillar during the mining of Panel 3n. Since no mining occurred in Panel 3s during this period, very little additional fracturing would have occurred ahead of the 3s face but some dilation would have occurred on the pillar. This could explain the measured ride during this period. As the elastic MinSim model showed zero strike ride, dilation and resultant buckling or shear on Prandtl-type fractures appear to be a reasonable explanation for the measured inelastic closure.

The closure profiles on both sides of the pillar suggest that the rigidity of the foundation system is soft and unable to effectively transmit load (York et al, 1998). York et al (1998) also suggest that the discontinuities required for the development of this condition should be less than 1 m below the surface. Thus, at least some of this fracturing was probably shallow-depth.

4.4.4 Discussion

Foundation damage was apparent at all three instrumentation sites. Extensometers, borehole camera surveys and the rods that were cemented into the pillars at the Amandelbult site, show a higher percentage of damage in the footwall than in the hangingwall at all three sites. At the Impala site, in particular, fracturing was not observed in the hangingwall above the 7s panel and, therefore, all the measured damage appears to have occurred in the footwall at that site.

One possible explanation for the observed preferred footwall damage is stress 'channelling', where the horizontal stresses were higher in the footwall than in the immediate hangingwall. Stress 'channelling' refers to the phenomenon of higher horizontal stress conditions in some lithologies than in others (Section 4.3).

Fracturing occurs parallel to the excavation when the horizontal stress conditions are high enough and the vertical confinement is relieved due to mining. Evidence of relatively high horizontal stress was observed in core retrieved from the footwall ahead of the stope face at the Impala site and possible discing in the Pseudo Merensky at the Union site. Comparatively higher horizontal stresses were also measured within the anorthositic stratum of the hangingwalls at both the Impala and Union sites. However, only the Amandelbult and Impala sites had an anorthosite footwall and the Amandelbult anorthosites may not have had a higher horizontal stress condition. Elastic MinSim models suggest that the critical strain criterion for failure and fracturing (Watson et al, 2005a) was exceeded in the footwalls at both the Impala and Union sites but that extension fracturing at the Amandelbult site was less likely. It would appear, therefore, that other factors might have contributed to the preferred footwall fracturing. The elastic strain at the lowest (limiting) depth of the observed fracturing at the Impala site suggested that the critical strain criterion might be closer to 0.24 mm/m than the 0.2 mm/m described in Watson et al (2005a).

At both the Amandelbult and Impala sites, the anorthositic footwall was more brittle than the pyroxenitic hangingwall and therefore more susceptible to fracturing. The results of laboratory tests on these rock types are shown in Figure 4-86. The results provided in Figure 4-86B were from samples tested in an MTS, servo-controlled machine at relatively low confinement (2 MPa). Figure 4-86A may be less reliable than Figure 4-86B because the test was conducted in a press that was developed in-house and might not have responded fast enough to the sample failure. However, both the results in Figure 4-86A and B show that the footwall anorthosites were more brittle than the hangingwall pyroxenites. The results of pillar dilation might, therefore, have caused greater damage to the pillar footwall than to the hangingwall in the pillar foundations. Since the pyroxenites generally have a higher modulus than the anorthosites, the footwall damage is likely to occur as a result of post-failure dilations. The gully cut adjacent to the pillar at the Amandelbult site would have helped to relieve the induced horizontal stress resulting from the dilating pillar, and promoted shearing of the footwall into the gully.

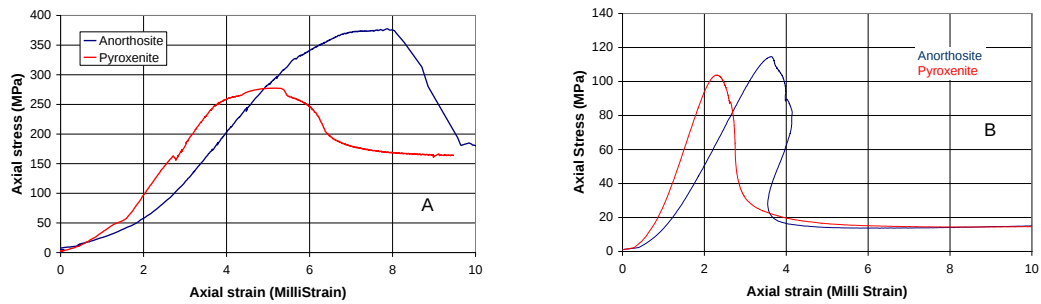


Figure 4-86 Laboratory tests showing post-failure behaviour for immediate hangingwall pyroxenite and footwall anorthosite from Amandelbult (A) and Impala (B)

While comparative brittleness may be a reason for the preferred damage of the footwall at the Amandelbult and Impala sites, the immediate footwall and hangingwall at the Union site was the same (pyroxenite). However, the instrumentation still showed that most of the inelastic closure resulted from footwall fracturing. The mechanism involved in the preferential footwall fracturing is more likely to be dominated by the tectonic stress condition at this site. As boreholes were not drilled into the footwall at the Amandelbult site, the stress condition of the footwall is unknown.

The timing of the blast holes may also have contributed to the damage in the immediate footwalls as the boreholes closest to the footwall were always ignited first. This means that the hangingwall blast holes needed only to fragment the rock and force it into the cavity formed below it. The footwall blast holes were comparatively more confined as there was no cavity above them and the rock had to be forced into the cavity formed by the previous footwall blast hole explosion. Damage was, therefore, greater in the immediate footwall than in the hangingwall. In particular, fractures could be extended further in the footwall, being extended by the blast gases over a longer period of time than in the hangingwall. The almost elastic conditions in the down-dip panel at the Amandelbult site, though, suggest that blast damage may only aggravate existing extension damage rather than developing additional fractures.

The closure measured at the centre of the 7s panel at Impala was compared to the convergence provided by a MinSim model. The parameters used in the model

were determined from laboratory tests performed on the nonlinear material from the site assuming a virgin vertical stress condition of about 30 MPa. While a reasonable correlation between the elastic and measured closure was observed from a face advance of about 6 m, the early closure was less than that predicted by the model. The reason for the lower measured closure is probably due to the inability of the model to capture the nonlinear behaviour. In particular, the higher modulus of the rock that had not reached the critical stress levels where micro fracturing occurs, such as over the face, resulted in a lower closure being measured in this area.

The blast damage would have acted to increase closure, but the contribution by the blast damage appears to have been small. The results from both the Amandelbult and Impala sites, therefore, suggest a small contribution of blast damage to the measured closure.

Significantly more inelastic deformation was measured adjacent to the up- and down-dip abutments than at the centres of the panels at the Impala and Union sites. This fracturing occurred at the Impala site even when the vertical stress condition on the abutments was relatively low. Inelastic deformation was particularly evident at the Union site during and after pillar formation. These closure profiles were compared to FLAC modelling performed by York et al (1998). The modelling showed the rock mass behaviour in repeating panels between repeating rows of elastic pillars. The observed model behaviour was interpreted as being various forms of buckling. The underground behaviour correlated with the model, with shallow-depth fractures/discontinuities with little ability to transmit load resulting in shallow-depth buckling. It appears that the loading of the abutments had a similar effect on the weakened footwall as the pillars did in the model. While the closure profiles from the Impala site were suggestive of general footwall buckling, Prandtl-wedge-type fracturing may have been responsible for the additional closure adjacent to the abutments and pillars. Both Prandtl-wedge- and Hertzian-crack-type fractures were observed below a pillar at the Impala site.

Another possible contributing factor to the higher closure adjacent to the abutments is the additional fractures that may have developed here as the stress

on the abutments increased. In reality, several factors probably contributed to this higher closure and the dominance of one factor over another may also have changed during the progression from abutment to pillar.

The shear stress around the advancing face, abutments and/or pillars – particularly during and just after pillar failure – may have caused shearing on some of the shallow-dipping fractures/discontinuities in the footwall. At the Impala site, most of the closure-ride stations around the pillars were installed after pillar failure and ride was not detected. The station at the centre of Panel 7s also showed that the ride after pillar failure was approximately elastic. Therefore, any shearing that occurred adjacent to the pillars could only have happened during pillar formation and early stages of pillar failure. However, evidence of lateral movement was shown by a shallow-dipping fault about 1.3 m below the footwall in the travelling way (Figure 4-87). The upper part of the discontinuity (immediate stope footwall) appears to have moved about 60 mm in a direction away from a large stability pillar, located about 5 m from the travelling way. The ride measured adjacent to the pillar at the Union site showed extensive footwall ride away from the pillar during and after pillar failure. The changes in ride also correlated well with the pillar dilation at the site, suggesting an association between footwall ride and pillar dilation. Slip on shallow-dipping discontinuities would explain why open discontinuities existed in the footwalls of both the Impala and the Union sites. Large volumes of water were lost at about 2.5 m below the stope during borehole drilling at both sites, suggesting extensive open planes at this depth. Slip and ride over asperities could also explain some of the inelastic closure that occurred.



Figure 4-87 Approximately 60 mm lateral deformation in the strike direction, observed on a shallow-dipping fault plane in the travelling way at Impala. A dip pillar was cut adjacent to this excavation

It is doubtful if geological discontinuities were responsible for the preferred footwall damage at the Amandelbult or Impala sites. However, such discontinuities could have influenced pillar punching at the Union site.

FLAC modelling showed that fractures can continue to develop in the foundation even after a pillar has reached residual strength conditions (Figure 4-88). The model assumed a slow but constant loading rate and the fracturing probably resulted from the lateral forces generated by the dilating pillar. Such fracturing does appear to have occurred at the Union site, until the loading dropped to a very low rate and pillar dilation stopped. Since the footwall extensometer measuring deformation below the panel at the Impala site was installed at a late stage of mining, additional fracturing was not measured at this site.

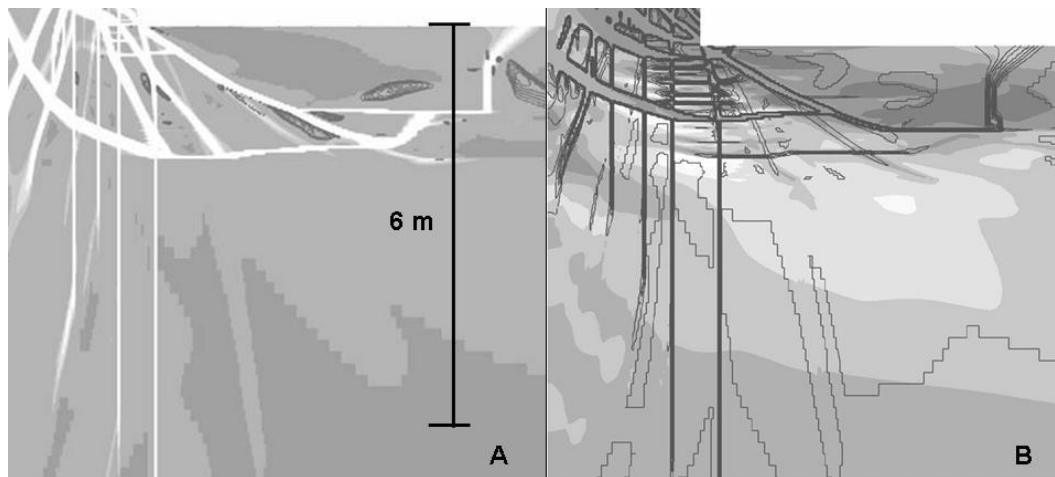


Figure 4-88 *FLAC fracturing in the foundation and pillar after 80 millistrains (white in A) and 125 millistrains (black in B)*

4.5 Summary

4.5.1 Hangingwall

The literature suggests that the formation of beams is common on the Bushveld and that beam formation often occurs through excavation-parallel stress fracturing of massive ground (Swart et al, 2000). Beam formation was observed in the immediate hangingwall of the Amandelbult and Union sites. The top and bottom of beams generally coincided with a change of rock type, even though these changes are gradational. In some instances at the Amandelbult site, stress profiles of beams were measured but no partings were observed. These profiles may be the result of interactions between different material types.

Conventional Voussoir beams (as described by Ryder and Jager, 2002) did not form at the Amandelbult site because of the relatively high k-ratio in the hangingwall. However, the k-ratio in the pyroxenites at the Union site was significantly lower. Blasthole sockets (Watson et al, 2005a) located in the declines below the Merensky Reef seem to suggest that a k-ratio of 0.5 is characteristic of the pyroxenites in that area. The horizontal measurements in the pyroxenites above site 2 also correlated well with an elastic model when a k-ratio of about 0.5 was assumed. This low horizontal stress condition is conducive to the formation of Voussoir beams and observations suggest that such beams could have formed in the immediate hangingwall.

Case studies on hangingwall behaviour show that hangingwall partings can occur well above the theoretical tensile zone. Roberts et al (1997) and Du Toit (2007) found open discontinuities as high as 25 m and 30 m over stopes with minor spans of between 70 m and 120 m. Similarly, unexpectedly high parting planes were observed up to 4.5 m in 30 m- and 35 m-wide panels at Union Section (Watson, 2003 and Du Toit, 2007). Elfen modelling showed that the vertical tensile zone height is adjusted when partings open, which would allow other partings to occur above the theoretical zone. This adjustment appears to be particularly applicable in blocky ground conditions. However, some open

discontinuities have been observed high above stopes in relatively good ground conditions; e.g. open discontinuities were observed up to about 20 m above 30 m-wide Merensky panels at Impala Platinum (Fernandes, 2007). These discontinuities were located in anorthositic rock types and suspected to be extension fractures. A similar strata-parallel parting was observed at 10 m above the stope at Union Site 2. The plane was located well above the 5 m-high vertical tensile zone (determined by a MinSim model) and could not be explained by a vertical tensile zone adjustment. However, it was located in a zone of high horizontal stress conditions. Therefore, it appears to be an extension fracture that developed when the vertical confinement was relieved during stoping operations.

The rock mass conditions at the Union site were evaluated as poor (blocky), with four joint sets and a high propensity for wedge and keyblock development. During mining, movement occurred on shallow-dipping and steeply dipping joints to greater heights than predicted by the height of the tensile zones. For example, open shallow-dipping discontinuities were observed to a height of 3.2 m at a span of 12 m. The MinSim elastic model suggested a vertical tensile zone of 2 m for that span. When the stope span was increased to 48 m, this height increased to 12.8 m and the predicted vertical tensile zone was only 5.8 m. However, no movement was observed at the base of the Bastard Reef and most of the inelastic deformation appears to have occurred within 3.4 m of the stope. This height coincided with the top of the immediate hangingwall pyroxenite lithology. A similar parting plane was observed in the top contact of the pyroxenite at a height of 5.6 m at Site 2. These fractures appear to have developed where the lithology changes. The other shallow-dipping fractures in the pyroxenites probably occurred on weak planes formed through igneous intrusion/extrusion or metamorphic flow processes, as suggested by the literature.

The heights of the predicted vertical tensile zones at the Union sites were probably altered by the open discontinuities, as indicated by the Elfen models. However, this height adjustment could not account for all the open discontinuities. In particular, the model could not account for the relatively great height at which steeply dipping joints were open. At Site 1, for instance, these discontinuities were open up to a height of 10 m at a span of 12 m (Appendix E). This was well above the height of the horizontal tensile zone if a k -ratio of 0.5 was assumed.

The additional height is explained by a mechanism of squeezing and shearing that is likely to have occurred along adversely oriented discontinuities. It is suggested that this could have occurred at the time when the face was advanced below the borehole position. The shearing probably influenced the stress condition, thus causing fracturing and horizontal parting planes to open above the vertical tensile zone. The survey site was located about 5 m horizontally from the north abutment, which remained stationary while the panel on the south side of the raise was mined. Further shearing/squeezing may have occurred in the hangingwall as the stress on this abutment increased. However, the original height of the open vertical discontinuities did not change, even when this abutment was mined away from the borehole. Since the span increased, the height of open discontinuities should also have increased, unless the abutment was the cause of the high open discontinuities. The measured deformations in the hangingwall were less than in the footwall, in the ratio of about 44% to 56%.

The borehole camera surveys at the Union sites suggest that in blocky ground conditions the gravitational effects on parting height may be increased due to small-scale shearing on adversely oriented joints when the face is advanced below. Further research is required to properly understand the behaviour of a blocky hangingwall condition.

Horizontal stress measurements performed above the centre of panels at both the Impala and Union sites suggested a k-ratio of about 0.5 in the pyroxenites. The measurements in the immediate stope hangingwall and in the Bastard Reef at the top of the measurement boreholes fitted well with elastic models when a k-ratio of 0.5 was assumed in the models. However, at both sites, the stresses in the anorthositic rocks between the two pyroxenite lithologies were significantly higher and corresponded to a higher k-ratio. This finding suggests that the pyroxenites may have a different tectonic stress condition to the anorthositic rocks at these sites. Variations in k-ratio, and in particular the high stresses measured in the lighter (anorthositic) rocks, suggests some form of stress 'channelling' in these rock types. However, this phenomenon was not observed at the Amandelbult site. Stress 'channelling' appears then not to be universally present in the anorthosites but may be an explanation for the high degree of fracturing observed in some leuconorites and anorthosites.

The hangingwall conditions at the Impala site were stable even though there was evidence that the rock mass had become nonlinear in locations where the stress had been sufficiently relieved. The many shallow-dipping and vertical boreholes drilled into the hangingwall above the panel showed that large-scale/macro fracturing only occurred adjacent to and above the pillars in the hangingwall. No partings were observed above the 7S panel even at the base of the Bastard Reef. However, approximately 4.7 mm of inelastic deformation was measured above the pillar just after pillar failure and a maximum of only 2 mm was measured in the footwall. The larger hangingwall deformation probably occurred because of the comparatively lower confinement (k-ratio) in the immediate pyroxenite hangingwall than in the footwall anorthosites. Shallow-dipping boreholes drilled over the pillars showed that vertical fracturing had developed to a height of about 1.2 m above the pillars, but no fractures developed above the stope. The more ductile nature of the pyroxenite could have restricted the hangingwall damage to the region above the pillar. The vertical borehole drilled down into the footwall from the panel (about 5 m down-dip of this pillar) showed that the footwall was heavily fractured, but this fracturing may not have been caused directly by the pillar.

The behaviour of the rock mass at the stress change measurement points above the pillars at the Amandelbult and Impala sites was linearly elastic. The only deviations occurred at the Union site, where the peak stress was slightly greater than the model, and the Impala site when the vertical stress dropped below about 10 MPa and the rock mass became nonlinear. The measurements at the Union site were only slightly higher than indicated in the elastic model, and this could have been the influence of the blocky rock mass on stress distributions.

Vertical stress change measurements conducted ahead of the face, and at the centre of Panel 8s at the Impala site, also showed that the rock mass behaved in a linearly elastic manner until the face had advanced about 8 m ahead of the instrument. At this point the vertical stress had dropped to about 10 MPa and significantly greater than linearly elastic strains were measured. It appears that this vertical stress was low enough for the opening of micro fractures and nonlinear elastic behaviour ensued. This additional strain was also accompanied by a drop in modulus.

Both the stress change and extensometer measurements showed good correlation to MinSim elastic models when the 'matrix' elastic constants, determined on intact laboratory samples, were applied. The investigation results suggest that the 'matrix' elastic constants derived on relatively small intact laboratory samples provide reasonable estimates of the rock mass constants prior to rock failure, or the development or opening up of discontinuities or fractures. The joint filling was generally thin at the measurement sites and a thicker filling might have altered the rock mass modulus. The extensometer at the Union site showed that once fracturing or opening of geological discontinuities occurred, the effective modulus of the rock mass dropped as expected.

The horizontal stress change measurements made above the pillars in a direction perpendicular to the long axis of the pillar or line of pillars was always greater than the elastic model during and after pillar failure. In addition, the measurements always showed an increase in horizontal stress after pillar failure even when the MinSim models suggested a decrease in this stress. This increase in horizontal stress was particularly noticeable at the Union site, where significant increases in horizontal stress occurred for some time after pillar failure. The reason for the deviations from the elastic models is probably the development of fractures in the hangingwall above the pillars. These fractures were observed to be parallel to the long axes of the pillars.

At the Impala site, a horizontal extensometer was installed over a pillar and between the edge of the pillar and the centre of the panel. The orientation of the instrument was approximately on dip and ranged in height between 1.5 m and 5.5 m above the stope. Prior to pillar failure this extensometer showed similar deformations to those predicted by an elastic model when the 'matrix' constants of the hangingwall material were used in the model. This good correlation was particularly noticeable over the pillar. During pillar failure, the extensometer over the stope showed compressional strain, indicating an increase in horizontal stress. At the same time dilation was measured at a height of about 0.9 m over the pillar. These results also suggest that the development of discontinuities over the pillar might have increased the horizontal stress over the stope, particularly near the pillar. The development of beams over the 8s panel might also have resulted in an increased horizontal stress in some locations of the hangingwall

near the pillar. These beams may have contributed slightly to the increased stress adjacent to the pillars but the probable induced horizontal stress over the stope would have been mainly affected by foundation fracturing over the pillar. A set of 2D FLAC models confirmed that fracturing above pillars increases horizontal stress in the hangingwall, particularly adjacent to the pillars. Steeply dipping fractures were seen above the pillars where the fractured edges of the pillars had fallen away at the Amandelbult and Impala sites. Vertical fractures were also observed to a height of about 1.2 m above the pillars during drilling operations at the Impala site and significantly higher fractures were apparent over the highly stressed stability pillar at the Amandelbult site.

The considerable increases in horizontal stress that occurred above the pillar at the Union site, after pillar failure, appear to have subsequently dropped to the same level as the vertical stress. The reason for this drop can be explained by creep that occurred as a result of block sliding in the blocky hangingwall around the pillar. Subsequent hangingwall behaviour over this pillar appears to have been approximately hydrostatic. The final horizontal stress condition suggests that the fractured rock above a pillar may behave in an approximately hydrostatic manner, particularly in blocky ground conditions.

4.5.2 Footwall

The instrumentation showed that the greater proportion of the damage due to mining occurred in the footwall of the stopes at all three sites. This was true even where the immediate hangingwall rock was weaker than the footwall. The immediate hanging- and footwalls of the Merensky Reef are generally pyroxenite and anorthosite, respectively.

All of the measured inelastic deformation appears to have occurred in the footwall at the Impala site as no fractures were observed in the hangingwall, except above and immediately adjacent to the pillars. Similarly, the extensometers at the Union site showed that about 66% of the closure was associated with inelastic footwall deformations at that site. Most of the inelastic component appears to have occurred during and immediately after pillar failure. The effects of gravity and creep were also evident on the hangingwall blocks during a period of no

mining and when the abutment closest to the extensometer (3n) was mined. At the Amandelbult site, the rotation of the lower rods that were grouted into the centre of both the monitored pillars indicated that the pillars were punching into the footwall.

From the instrumentation results, it appears that there are four possible reasons for this preferred footwall fracturing:

- comparatively higher horizontal stress conditions in the immediate footwall;
- distribution and orientation of geological structures such as joints and faults;
- comparatively greater brittleness of the footwall material; and
- blasthole timing to protect the hangingwall against damage.

The stress measurements in the hangingwalls of the Impala and Union sites showed comparatively higher horizontal, tectonic stresses in the anorthositic rocks than in the pyroxenitic rocks (stress 'channelling'). The theory suggests that fractures develop parallel to the stope when the horizontal stress is sufficiently great and the vertical confining stress is reduced or relieved during mining.

Evidence of comparatively higher horizontal stress was observed in the immediate stope footwall at the Impala site, when a vertical borehole was drilled ahead of the face. At this location, discing was observed in the immediate stope footwall but not in the hangingwall. Although the extent of the mining influence on the magnitude of the stresses is not known, the concentrations would have been similar for both the foot- and hangingwall. High horizontal stress in the footwall was also suggested by the apparent discing observed in core drilled through the Pseudo Merensky at the Union site. Stress 'channelling' was, however, not observed everywhere in the anorthosites, e.g. the hangingwall at the Amandelbult site. There is, therefore, a high probability that the immediate anorthosite footwall did not have a higher horizontal stress than the pyroxenite hangingwall at that site either. Thus the tectonic stress condition appears to account for the preferred footwall damage at only two of the three sites.

Elastic models suggest that the closure profiles should have reduced between the centre and the boundaries of the excavations (abutments) during isolated mining conditions. However, a similar closure was measured across the panels at both the Impala and Union sites, with a slight increase in closure adjacent to the abutment at the Impala site. The faces and abutments at both sites were almost unfractured at the time, indicating relatively low vertical stress conditions on the abutments when the footwall fracturing occurred. Significant fracturing was also observed in boreholes drilled into the footwalls of both the Impala and Union sites. In particular, fracturing was observed in certain lithologies to about 3 m below the footwall at the Union site when the ledging span was only 12 m. The fracturing at these sites can be explained by some sort of extension fracturing (Stacey, 1981) that occurred when the vertical stress was relieved by mining. The slightly greater fracturing near the abutment could have occurred as a result of the stress trajectories around the abutments being added to an already high horizontal stress or the effects of pillar/abutment dilation.

Laboratory tests performed by Watson et al (2005b) suggest that the critical strain necessary for extension fracturing in anorthosite may be 0.2 mm/m. Elastic MinSim models showed that this strain was exceeded in the footwalls at both the Impala and the Union sites but not at the Amandelbult site. The formation of the observed horizontal fractures at a span of 12 m (Union site) is difficult to explain unless the extension strain criterion is assumed.

An elastic strain of 0.24 mm/m was determined for the level at which the deepest (limiting) fracturing was observed at the Impala site. Assuming the stresses determined from the blasthole sockets were correct, the critical strain in the anorthosites may be closer to this strain than that suggested by the laboratory tests in Watson et al (2005b).

The extensometer at the Union site showed that prior to the pillar failure, the inelastic deformation was mainly restricted to 5.5 m below the stope. However, after pillar failure, deeper-level fracturing occurred. The mechanism can be explained in two ways:

- Extension fracturing, because deeper-level fracturing occurred when the confining effects of the pillar on the deeper levels was reduced at pillar

failure. A similar extension fracture was observed in the leuconorites in the hangingwall above the stope after the confinement was relieved during mining.

- Additional stress was transferred to the nearby abutment and the stress trajectory around the abutment was added to an already high horizontal stress.

The second explanation is unlikely as the stress transfer would probably have been relatively small. However, the two mechanisms could have worked in tandem to cause an increase in fracturing.

Since no boreholes were drilled into the footwall at the Amandelbult site, there is no information on footwall geological structures at this site. However, FLAC models show that pillar stress regeneration only occurs under minimal foundation damage. Thus the observed stress regeneration suggests that the influence of such features was minimal.

Spencer and York (1999) measured significantly more deformation in the strata above a parting plane some 4 m in the footwall at another instrumentation site at Impala Platinum. During the current research, such a prominent plane was observed about 1.3 m below the 8s panel. Some lateral deformation of the rock above this plane was apparent in a direction away from a large pillar. This would have influenced the closure. However, very few discontinuities were observed in the footwall of the 7s panel and FLAC modelling suggests that a single fault plane with a friction angle of about 30° is unlikely to influence pillar behaviour unduly. It is therefore unlikely that geological discontinuities are the main cause of the preferred footwall damage at this site. However, it is possible that such discontinuities influenced pillar punching at the Union site.

The preferential damage of the anorthositic footwall at the Amandelbult site may have been due to the easier fracturing of the comparatively more brittle footwall rock as a result of pillar/abutment dilation. Fractures probably developed below the abutment prior to pillar formation, i.e. ahead of the down-dip face. (These fractures have been observed at other sites where they were exposed during subsequent footwall excavations). The comparatively higher inelastic and almost elastic closure measured on the up- and down-dip sides of the pillar, respectively,

might have been the result of mobilisation of existing fractures on the up-dip side of the pillar. The development of new fractures would thus have been retarded as the potential for fracture development was reduced by slip on existing fractures. The closure measurements showed that these fractures probably extended below the ASG, to about 1.5 m below the stope. Again, this damage could not have been severe as FLAC models indicated that the observed pillar stress regeneration is only possible where little or no foundation damage has occurred.

The development and orientation of fractures on the up-dip side of the pillar at the Amandelbult site may have been influenced by the presence of the ASG. Curved fractures have been observed elsewhere under a 2 m-wide siding between an abutment and a gully. At that site, fractures only developed in the anorthosite footwall and terminated on a 1 cm-thick chromitite band between the anorthosite and pyroxenite in the abutment. Also, horizontal boreholes drilled vertically above each other in an abutment at the Impala site – one in the anorthosite and the other in the pyroxenite rock types – showed horizontal borehole breakout in the anorthosites but not in the pyroxenites. This occurred even though the horizontal, confining stress in the pyroxenite was probably lower. Since the boreholes were in the same abutment, the vertical stress should have been the same. These observations suggest that fractures develop more easily in the anorthosites than in the pyroxenites. Laboratory tests showed that the anorthositic footwall of the Amandelbult and Impala sites is more brittle than the pyroxenite hangingwall. However, the footwall was not more brittle than the hangingwall at the Union site and more damage was also observed in the footwall at this site. In reality, the preferred fracturing/damage of the panel footwall is probably a combination of variable components of several mechanisms.

An additional influence to the fractured condition of the footwall may have been the timing of the blast holes. The boreholes closest to the footwall were always ignited first. Under these confined conditions, the blast fractures developed further than in the upper unconfined holes. Thus the blast damage would have been more severe in the footwall than in the hangingwall, being driven by the blast gases for a longer time in the footwall. The almost elastic conditions on the down-dip side of the pillar at the Amandelbult site, though, suggest that the blast damage may be restricted to enhancing damage associated with existing

fractures. The closure results from the Impala site also suggest that the blasting effects did not have a major influence on closure at this site, as less inelastic deformation was measured close to the face than further back (assuming a modulus of 15 GPa in the model).

Very little ride was observed adjacent to the pillars after pillar failure at the Impala site. The station at the centre of Panel 7S also confirmed that no inelastic ride occurred after pillar failure. However, some ride may have occurred prior to the installation of the pillar stations. Large quantities of drill water were trapped in certain discontinuities in the footwall at both the Impala and Union sites, indicating that these discontinuities were open and probably extensive. The extent of these fractures suggests that a small amount of lateral movement probably occurred on them. Lateral movement was also apparent on a strata-parallel fault structure, 1.3 m below Panel 8s.

The strike ride measured adjacent to the dip-oriented pillar at the Union site showed a direct relationship between footwall ride and pillar dilation, during and after pillar failure. Since no strike ride was predicted by an elastic model, the results are inelastic and suggest lateral shearing and/or development of fractures. This deformation may have been assisted by the presence of the centre gully about 4.5 m distance from the pillar. Both the ride and closure results at the Union site suggest that pillar punching, foundation damage and possibly buckling may have occurred during and after pillar formation. Since ride measurements were only made adjacent to the pillar and not close to the centre gully, the real effects of the gully on the pillar behaviour could not be determined. However, a similar and significantly smaller closure was measured at 2 m and 3 m from the pillar edge, suggesting that most of the inelastic deformation probably occurred immediately adjacent to or under the pillar. Very little footwall ride was measured after stoping had been completed, and this coincided with a significant reduction in pillar dilation.

The ride that was measured on dip (parallel to the long axis of the pillar) could be simulated by an elastic model if a modulus of 9 MPa was used in the model. This model also simulated the closure at the measurement position, suggesting that

the fractured rock mass behaved in an approximately elastic manner. However, a very low modulus was required to compensate for the fractured rock.

The closure profiles on the other side of the pillar, in Panel 3s at the Union site, also suggested large-scale inelastic deformations adjacent to the pillar and final face position. These measurements were made well after pillar failure, when the vertical stress on the pillar varied between 32 MPa and 47 MPa. At this stage the face had also reached its final position. The measurements suggest that dilation of the pillar and abutment probably resulted in buckling of the fractured footwall or shear on Prandtl-type fractures.

An extensometer was installed through the centre of a pillar at the Impala site and deformations were measured in the immediate foot- and hangingwall of the pillar. Prior to pillar failure the footwall and hangingwall behaved in a linearly elastic manner with the same elastic constants as determined on a laboratory sample that had not been affected by micro-fracturing. A direct comparison between the footwall and hangingwall measurements suggested that the whole pillar shifted upwards by about 1 mm, i.e. hangingwall punching. However, the punching effect of the pillar on the extensometer readings would have had an opposite effect (compression) to any fracturing that may have developed in the footwall. The two mechanisms of inelastic deformation could have worked against each other and it is impossible to determine the contributions of each in the measured deformations. In addition, a 10 mm-diameter cable was sheared off in the borehole just after pillar failure. This shearing probably resulted in additional, apparent extension deformation. Unfortunately, the amount of shearing cannot be established from the results, but was probably small since no ride was measured from the time the closure-ride stations were installed. The extension deformation would have been less than the elastic model if the measurements included only the effects of the punch.

The lack of fracturing over the stope suggests that Prandtl wedge-type fractures did not form in the hangingwall at the Impala site. The comparatively greater initial inelastic hangingwall deformation probably occurred due to a lower horizontal confining stress in the pyroxenites. The more ductile pyroxenites were probably also better able to accommodate the lateral deformations that resulted

from the initial foundation failure, than the more brittle footwall anorthosites. Thus the hangingwall failure was restricted to the rock above the pillars, whereas the final pillar behaviour was controlled by the footwall damage that extended into the stope. Unfortunately the extensometer readings after pillar failure were unreliable as the hole appeared to have sheared.

A reasonably good correlation between the elastic numerical model results and the closure measurements was observed at the centre of the isolated panel at Impala when a low modulus of 15 GPa was employed in the model. This was particularly evident for face advances between 7 m and 22 m. The low modulus was used to account for the nonlinear behaviour that was determined from the laboratory tests, assuming vertical virgin stress conditions of about 30 MPa. The good match between the low modulus model and the measurements suggests that the nonlinear material was adequately represented by the model over these face advances. However, more convergence was estimated by the elastic model than actually measured during the first 7 m of face advance. Three possible reasons are advanced for the early mismatch:

- the inability of the model to simulate a separate modulus over the face where the material was linear and therefore much stiffer;
- the inability of the model to simulate extension fracturing – the discing in CSIRO1 seems to confirm the relatively high horizontal stresses required for this type of fracturing; and
- convergence occurring ahead of the face as a result of face crushing.

The model assumed constant, relatively low linearly elastic constants across the stope. In reality, however, in all zones where the vertical stress was higher than about 10 MPa, the rock had not micro fractured and had a comparatively much higher modulus than assumed in the model. These areas should have had a smaller influence on closure than predicted by the model and include the immediate face and abutments.

The difference in fracturing observed in a vertical borehole drilled into the footwall behind the face and the core from a vertical footwall borehole drilled ahead of the face showed that significant fracturing developed in the footwall during mining at

the Impala site. Once developed, these fractures may not have had a significant effect on the closure measured away from the face.

It is unlikely that significant closure occurred ahead of the face at the Impala site as the typical, steeply dipping extension fractures were not observed in the hanging- or footwall. In addition, the face and the abutments adjacent to the face were almost unfractured at the time of installing the closure meters.

The profile of the closure measurements across the dip span at the Impala site indicates additional, inelastic closure adjacent to the abutments. A slightly larger closure was measured here, even though the contributions of the stiffer, linear rock over the abutments were not considered in the elastic model. A similar trend of comparatively high closure adjacent to the pillar/abutment was measured at the Union site. The difference between the modelled and measured closure profiles indicated that significant inelastic closure occurred at this site. The two closure stations at the edge of the pillar at the Union site showed that closure also varied along the length of the pillar/abutment.

The modulus required for the numerical simulation of the measurements at the Union site was much lower than that determined in the laboratory, suggesting that fracturing probably occurred across the whole span. However, the comparatively larger closure measured adjacent to the pillar and the similar but smaller closure measured at 2 m and 3 m from the pillar suggest that the influence of the pillar on closure may have been mainly in the immediate vicinity of the pillar at the Union site.

FLAC modelling was performed by York et al (1998) to simulate the rock mass behaviour around panels in a stope of repeating panels between repeating rows of elastic pillars. The model behaviours were interpreted as being various forms of buckling influenced by the depth of a parting plane. A comparison between the closure profiles from the Impala and Union sites and this modelling suggested the buckling of a soft footwall with a shallow-depth parting plane. It appears that the loading of the abutments may have had a similar effect on the weakened footwall as the pillars did in the model.

The original interpretation of the FLAC modelling may, however, have been wrong and the supposed buckling in the models could have been deformation resulting from Prandtl wedge-type fracturing or fracture development parallel to the stress trajectories. Both Prandtl wedge- (Prandtl, 1921) and Hertzian crack- (Hertz, 1896) type fractures were observed below a pillar at the Impala site.

The shear stress around the advancing face may have caused shearing on some of the existing shallow-dipping discontinuities. High stresses around the pillars during and just after pillar failure may also have caused shearing on appropriate shallow-dipping discontinuities. Shearing of pre-existing discontinuities would result in ride over asperities, thus opening the discontinuities and contributing to closure. In addition, shearing would aid in the development of longer fractures. Such fractures or discontinuities were suggested by the loss of water during the drilling of the footwall boreholes at the Impala and Union sites at about 2 m to 2.5 m below the stope. Evidence of lateral movement was also observed in the footwall travelling way at the Impala site (Figure 4-87).

FLAC models suggest that the foundation fracturing continues to increase after pillar failure if there is continued loading or deformation of the pillars. This condition was suggested by measurements for a few months after mining had been completed at the Union site. However, no additional fracturing was measured at the other two sites. The reason may be because the loading reduced to a very small rate once the panels had been mined out. Unfortunately the footwall extensometer in the stope at the Impala site was installed late, after mining had almost been completed in Panel 8S. These late measurements confirm that inelastic deformations did not occur in the footwall of Panel 7S for more than 12 months after stoping operations had been completed at the site. Similarly, no inelastic deformations were measured in the hangingwall of this site, even at the base of the Bastard Reef.

The results of the research suggest that several factors are contributing to the preferred footwall fracturing and related inelastic closure. At the Impala and Union sites, the dominant mechanism appears to be extension fracturing, which weakened the footwall and effectively reduced the pillar w/h ratio. The weaker footwall probably encouraged the development of Prandtl wedge-type fracturing

and may have facilitated buckling to accommodate pillar dilation. These deformations resulted in additional closure, which explains the comparatively higher closure adjacent to the pillars and abutments. A relatively more brittle footwall at the Amandelbult site probably resulted in fractures developing preferentially in this foundation during pillar dilation. Once the footwall had been weakened by the fracturing, pillar punching initiated. The comparatively more brittle footwall at the Impala site probably also facilitated damage in the foundations below the pillars at this site.

An excellent correlation was shown between the footwall extensometer results at the Impala site, prior to pillar failure, and a MinSim model with the laboratory-determined 'matrix' elastic constants. The stress change measurements in the hangingwall at the Amandelbult and Impala sites also correlated well with numerical models when these constants were applied. The results suggest that it is unnecessary to downgrade the laboratory results to account for the intact rock mass in good rock mass conditions. However, some adjustments to the elastic constants may be necessary when there are large numbers of discontinuities with significant thickness of soft fill material.

The research described in this chapter (Chapter 4) represents the first major/significant measurement of stress and deformation behaviour in the platinum mines of South Africa. The investigations have made major contributions to the understanding of rock mass behaviour around pillars and stopes on the Merensky Reef. In particular, the concepts of stress 'channelling', nonlinear elastic rock mass behaviour and fracturing around the pillars and panels have not been extensively studied in the context of the platinum mines. A database of measurements and observations has also been created that can be used for further research and analysis in the future.

The next chapter (Chapter 5) deals with the behaviour of pillars. The field stress and stress change measurements conducted above the pillars at the three instrumentation sites described in the current chapter will be used to determine pillar stress. The evaluation of pillar strength, both peak and residual, is discussed together with pillar behaviour. The peak strength evaluation of pillars at the Impala site is checked against a back-analysed database of stable and failed

pillars from the Impala Platinum Mines, and strength formulae are provided. The residual strength of the pillars is compared to a series of field measurements that were conducted to determine a stress profile across the pillar. Since most of the underground measurements were confined to a narrow range of w/h ratios, the results were supplemented with laboratory tests and FLAC models to determine a residual strength-w/h ratio relationship. Finally, criteria for 'crush' pillar design are provided.

5 Pillar behaviour

Chapter 4 discussed measurements made to determine the rock mass behaviour around Merensky Reef pillars and stopes. The aim of the investigation was to determine the causes of the inelastic behaviour often observed in the panels and to understand the interaction between the pillars and the rock mass around the stope.

Chapter 5 describes pillar behaviour and shows the influence of foundation rocks on this behaviour. The chapter shows that the so-called “squat” effect, normally observed when pillars are significantly weaker than the foundation materials, does not dominate the pillar system behaviour when the pillar and foundation rocks are similar. As most of the research was conducted on ‘crush’ pillars a great deal of the chapter is dedicated to the pre- and post-failure behaviour of these pillars, including peak and residual strengths as well as design criteria. The work is based on underground measurements, and laboratory tests and numerical models aided in the interpretation of these measurements. The APS and peak pillar strength values quoted in the chapter were calculated from the stresses normal to the reef plane.

5.1 Introduction

York and Canbulat (1998) showed that the strength of a rock specimen is a function of the end, or boundary, conditions. These conditions include interface friction and the relative material properties and geometry of the loading platen/end piece/footwall and/or hangingwall (assuming other conditions such as loading rate, temperature and moisture content are unchanged). Strength is therefore not a stand-alone value, but is an assessment for a given set of boundary conditions. Thus, pillar strength should be quoted with a statement regarding the end conditions, the means of load control and loading rates. The author suggests that previous results have been presented with ill defined boundary conditions, especially for model pillar studies.

Chapter 4 showed that pillar failure extended into the foundation rocks. The current chapter will show that an understanding of the interaction between the pillar and its foundation is critical in pillar design. The study concentrates mainly on underground measurements of pillar stress and strain, with modelling and laboratory tests to clarify certain behavioural patterns. Most of the work was performed on pillars of about 'crush'-pillar size, which allowed measurements to be made of peak and residual strength as well as post-failure behaviour.

'Crush' pillars were introduced to the mining industry at Union Section by Korf (1978) to stop a serious backbreak (stope collapse) problem where at least three to four stopes were collapsing per month. Difficulties were experienced when stoping advanced to a point 30 m to 40 m on both sides of the centre gully. Sudden failure of the beam frequently occurred at this stage, resulting in parting of the rock at the bottom contact of the Bastard Reef some 20 m above the stopes. The pillars that were introduced had dimensions of 1.5 m x 3 m and a height of about 1 m. Although the pillars had obviously failed (crushed) near the working face, the introduction of these pillars stopped the stope collapses in the mining area where they were used. This area extended from 100 m to 700 m below surface and about 1300 m along strike. Today the use of these 'crush' pillars is widespread across the platinum industry in the form of small in-stope chain pillars.

These 'crush' pillars provided the author with a unique opportunity to study pre- and post-failure behaviour of Merensky pillars. Apart from evidence provided by Korf (1978), little work has been done to determine the residual strength of 'crush' pillars and there is concern that large-scale collapses could occur in deeper areas supported on such pillars. The original pillar widths have, therefore, been increased on some mines, and a significant range of w/h was available for investigation where undisciplined pillar cutting had taken place. A series of pillar bursts that occurred on the Merensky Reef in 2004 with serious consequences has raised questions with respect to 'crush' pillar design. These and other technical issues have been investigated in the current study.

5.2 Literature review: pillar peak and residual strengths

5.2.1 Peak pillar strength

Currently the rock engineers of most platinum mines estimate peak pillar strength using an adjusted version of Hedley and Grant's (1972) formula. This empirical formula was adapted for hard rock from the Salamon and Munro (1967) formula, originally designed for square coal pillars:

$$PS = Kh^{\alpha}w^{\beta} \quad 5-1$$

Where:

$K = 7.2$ MPa (strength parameter - $\alpha = -0.66$ (coal)
determined from a back-analysis)

h = Pillar height (m) $\beta = 0.46$ (coal)

w = Pillar width (m)

The Hedley and Grant (1972) modifications were based on a back-analysis of hard rock pillars from the Elliot Lake district in Canada and are tabulated below:

$$K = 0.67 \times \text{UCS} \quad \beta = 0.5$$

$$\alpha = -0.75$$

Some agreement appears to exist between this formula and visual observations of the Bushveld Merensky pillars when the K -value is modified in the range between $0.3 \times \text{UCS}$ and $1.0 \times \text{UCS}$. A means of determining this factor from a down-rated rock mass strength (DRMS) is described by Laubscher (1990). The method employs a rock mass rating to include the jointing effects of the rock mass.

It should be noted that the Salamon and Munro (1967) and Hedley and Grant (1972) investigations considered pillars with w/h ratios of 0.9 to 3.6 and 0.7 to 1.5 respectively. The formulae appear to underestimate pillar strengths for higher w/h ratios. In coal pillars this underestimation appears to exist for ratios greater than

about four and Salamon (1976) revised the coal formula to include all w/h ratios (Equation 5-2). The expression is commonly known as the “squat” pillar formula.

$$PS = K \frac{R_0^c}{V^d} \left\{ \frac{c}{e} \left[\left(\frac{R}{R_0} \right)^e - 1 \right] + 1 \right\} \quad 5-2$$

Where:

K = Rock mass strength

V = Pillar volume (m^3)

R = Pillar w/h ratio

R_0 = Critical w/h ratio

e = Rate of strength increase

c, d = Constants.

Wagner and Madden (1984) conducted laboratory tests on cylindrical specimens of sandstone to determine the constants in the “squat” pillar formula. Both R_0 and e were found to be 4.5. Stacey and Page (1986) suggested that R_0 and e are 4 and 4.5, respectively for coal. Merensky Reef pillars are different from coal pillars as the pyroxenite and anorthosite pillar material is usually similar in strength to the foundation rocks. Handley et al (1997) found that for hard rock pillars, seismicity associated with foundation fracturing and failure initiated when pillar stress levels exceeded $1.2 \times$ UCS of the foundation material. This finding suggests that the Merensky pillar strength could be strongly dependent on the properties of the foundation rocks.

A more recent design methodology for pillars at shallow depth was provided by Ryder and Özbay (1990). The methodology entails applying a series of correction factors to the UCS of the pillar material as follows: Factor F_1 deals with weakening effects of discontinuities in the rock mass (size effects). For good quality rock mass conditions, the size effect correction is:

$$F_1 = 0.4$$

The shape correction, F_2 , accounts for the strengthening effects of rectangular or rib-shaped pillars. The strength of a rectangular pillar is determined by multiplying the strength of a square pillar by an appropriate factor shown in Table 5-1. Two

correction factors are used in the w/h analysis. F_3 (~1.3) accounts for the strengthening effects of increasing the w/h of a laboratory specimen of 1/3 to a unit cube and F_4 corrects for some of the common w/h ratios of stope pillars (Table 5-2).

Table 5-1 Correction factors for pillar shape

Table 5-2 Correction factors for pillar w/h ratios

F_2	Pillar shape	F_4	w/h ratios of pillars
1,0	1 x 1	1,0	1
1,1	2 x 1	1,2	2
1,2	4 x 1	1,4	3
1,3	∞ x 1 (Rib pillar)	1,6	4

The suggested methodology includes the factors that influence a pillar itself but fails to provide for the effects of the foundations on pillar behaviour. In coal mines where the foundations are much stronger than the pillars and the loading system is relatively stiff due to the comparatively small panels or rooms, the abovementioned technique might provide good results. However, this system severely underestimates the strength of Merensky Reef pillars – e.g. the strength of a 5 m x 5 m pillar in a 1.2 m stoping width was measured at Impala Platinum to be 295 MPa, while the application of the above methodology suggests a strength of 83 MPa.

Pillars with $w/h > 10$ are essentially indestructible (Ryder and Özbay, 1990). However, foundation failure may be an issue. Brittle pillars resting on weak or plastic foundations can be weakened by induced horizontal shear/tensile tractions or loss of w/h confinement due to slip on weak partings. The pillar system as a whole can be weakened as a result of punching/shearing/heave in the foundations. Little quantitative information on the magnitude of this weakening appears to exist, although Kabeya (in York et al, 1998) did extensive modelling that indicated that the bearing capacity could vary between 1 and 5.2 times the UCS of the foundation material. The weakening effects appear mainly

to be associated with discontinuities, k-ratio and panel span. In addition, little is known about the influence of creep. A useful pointer to the potential for creep weakening may be Bieniawski's observation (1967) that creep onset occurs at the point in a uniaxial test where volumetric strain changes from compaction to a dilatatory behaviour.

York et al (1998) and Bieniawski and van Heerden (1975) suggest that the 'linear' w/h strengthening law (Equation 5-3) is superior to the traditional Salamon-Munro 'power law' formulation.

$$PS = K_i \left[b + (1-b) \frac{w}{h} \right] \quad 5-3$$

Where:

K_i = *In situ* cube strength

w = Pillar width (m)

b = constant

h = Pillar height (m)

Laboratory tests performed by York et al (1998) on Merensky Reef samples showed that the critical rock mass strength (*in situ*, large-scale strength) could be determined on relatively small specimens of about 150 mm in diameter. This advantage of determining large-scale strength from relatively small specimens has important implications for Merensky pillar design. Because no volume effect is expected above the critical size, the implication is that Merensky pillar design need not be concerned with the volume effect. A further implication is that the power formula, which is relatively difficult to derive, need not be used; rather, a simple linear function is applicable.

The critical strength of the Merensky Reef was predicted in laboratory testing by York et al (1998), with the use of cylindrical specimens with $w/h = 1$. The tests were performed between steel platens and showed a critical strength of about 110 MPa for samples from both Amandelbult Platinum Mine and Impala Platinum Mine. The back-analysis of 'crush' pillars from Impala Platinum Mine (Section 5.3) suggested a higher *in situ* cube strength of 136 MPa. The lower laboratory result can be expected as the steel-sample contact friction angle was only about 12°, which is significantly less than for the underground loading conditions.

York et al (1998) also determined the normalised strength effect of w/h ('1-b' in Equation 5-3) from laboratory tests between steel platens. The tests indicated this value to be about 0.24, which was significantly lower than the 0.59 shown by the back-analysis in Section 5.3. However, 2D numerical modelling showed that if the contact friction angle was increased from 13° to a more realistic 30° , the normalised strength effect would be about 0.5, which was closer to the back-analysis. The numerical models only explored the effects of contact friction and did not account for draping effects or foundation failure. An improved normalised strength effect could be determined from further laboratory work with more realistic boundary conditions.

Esterhuizen (1997) determined the effect of jointing on coal pillar strength through numerical modelling and back-analysis of existing pillars in the coal fields of South Africa. An example is shown for pillars of $w/h = 3$ (Figure 5-1). The y-axis shows the ratio of the strength of a pillar with jointing and a pillar with no jointing. The reduction in pillar strength is shown as a function of pillar joint dip angles and joint frequency, with separate curves drawn for different joint frequencies. Another method of accounting for jointing involves a material downgrading of laboratory strength according to the technique described by Hoek and Brown (1988). The very small standard deviation shown by the back-analysis in Section 5.5.6 indicates that the pillars in this database were almost unaffected by joints, probably because these discontinuities were few and almost vertical.

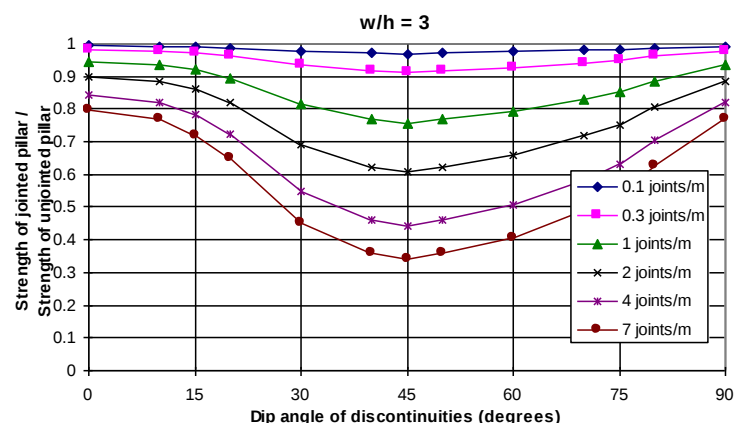


Figure 5-1 The effect of dip angle and frequency of jointing on pillar strength, for pillar $w/h = 3$ (after Esterhuizen, 1997)

A back analysis of pillar strength was attempted by Spencer and York (1999) for 3m x 6m Merensky Reef pillars at Impala Platinum. The research was based on field closure, deformation and stress measurements, laboratory testing and numerical back-analysis. The *in situ* pillar peak stress was inferred to be 150 MPa, which is significantly less than the 246 MPa suggested by the back-analysis described in Section 5.5.6 for the same size pillar. However, the Spencer and York (1999) analysis suggested that the pillar did not fail but yielded, maintaining a constant 150 MPa for the duration of the mining period. The paper shows a strong influence of the foundation on pillar behaviour, as the yielding and deformation in the panels could be associated with a weak clay layer and jointing in the immediate footwall. The findings suggest that the influence of the foundations is an important factor to consider when determining pillar behaviour and strength.

5.2.2 Residual strength of ‘crush’ pillars

Prior to the investigations discussed in this thesis, very few attempts had been made to measure the peak and/or residual strengths of pillars. The literature makes the following estimates of residual strengths of ‘crush’ pillars in the Bushveld Platinum mines:

- 5% to 10% of peak strength (Ryder and Özbay, 1990). These percentages translate to between 8 MPa and 20 MPa for UG2 and Merensky reefs.
- 19 MPa (Roberts et al, 2004). This result was based on 2D stress measurements above Merensky pillars.

Apart from these two estimates of residual strength, little is known about the post-failure behaviour of ‘crush’ pillars. Ryder and Özbay (1990) suggest that this is a problem on which further field and laboratory experimentation needs to be carried out.

Back analyses performed by Roberts et al (2005) at Northam Platinum Mine on the UG2 reef and Randfontein Estates Gold Mine suggested that about 1 MPa support resistance was sufficient to arrest a stope collapse or, as it is known colloquially, a backbreak. Partings that had opened in the hangingwall some

28 m above the workings at Northam Platinum Mine were stabilised by backfill once a stress of about 1 MPa had developed in the backfill. If an extraction ratio of 92% is assumed, pillar residual strength should be about 13 MPa. Since 'crush' pillars with w/h ratios of about 1.5 were shown by Korf (1978) to stop stope collapses, the findings of Roberts et al (2005) suggest that these pillars might have had a residual strength of greater than 13 MPa.

Ryder and Jager (2002) suggest that there is a potential for violent failure if the pillars are not crushed at or within a few metres from the stope face. Pillars that fail away from the face are subject to soft loading conditions, which increase the likelihood of a violent failure. Since the w/h ratio of pillars affects strength, it is important not to cut 'crush' pillars too large. Spottiswoode et al (2006) showed that a significant amount of the recorded seismicity at Impala 10-shaft resulted from violent failures of pillars that were cut too large. Some of these events caused damage to excavations and infrastructure.

In essence, 'crush' pillars should not be too large as the potential for bursting is high for w/h ratios between about 3 and 5 (Ryder and Jager, 2002). On the other hand, 'crush' pillars should not be too small as the residual strengths of these pillars should provide the required support resistance to prevent backbreaks and keep the immediate stope hangingwall stable up to prominent parting planes – e.g. the base of the Bastard Reef 15 m to 30 m above the Merensky Reef.

5.3 Influence of boundary conditions

The boundary conditions assumed in the modelling of a pillar can have a profound effect on the modelled pillar behaviour. Three scenarios with exactly the same input parameters were modelled, with the use of FLAC, to determine:

- the effects of draping of the hangingwall and footwall around a pillar in a stope (Figure 5-2), excluding foundation failure and without an interface between the pillar and the hangingwall or footwall;
- pillar behaviour without hangingwall and footwall draping, but with elastic loading foundations and without an interface between the pillar and the foundation; and
- pillar behaviour in a test laboratory with elastic loading platens and an interface friction angle of 15° .

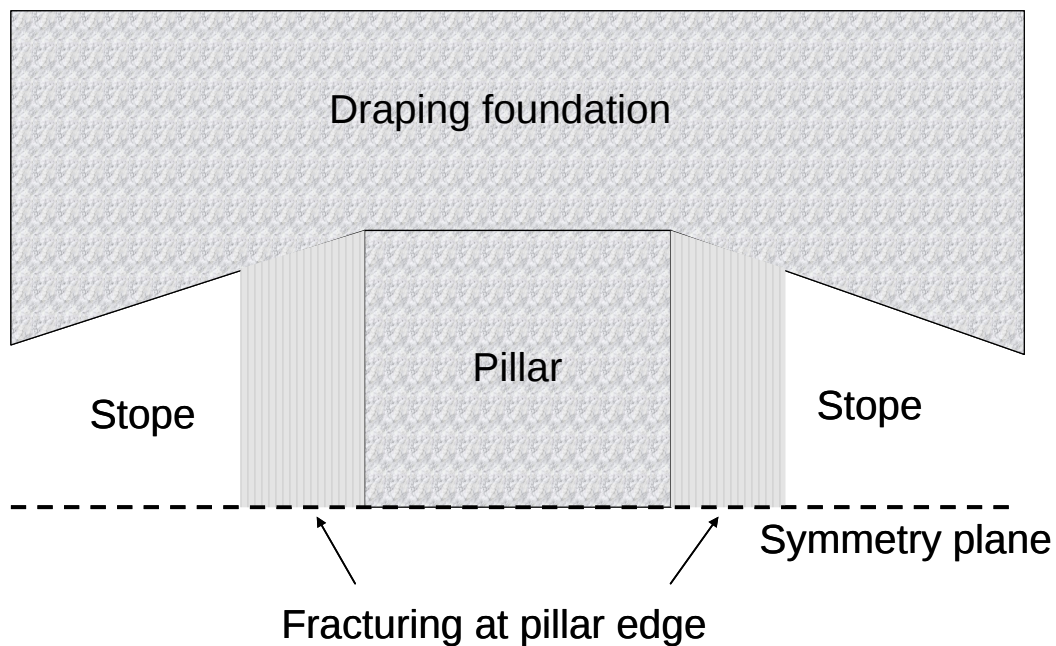


Figure 5-2 Diagram illustrating hangingwall or footwall draping

The three scenarios were modelled with the Mohr-Coulomb failure criterion and strain softening using the parameters in Table 5-3. A pillar with w/h ratio of 2.8 was modelled in all three scenarios. The results of the investigation are shown in Figure 5-3. In this modelling exercise the actual magnitudes of stress are of no

consequence as the models were intended only as a comparison; the strength parameters were, in this case, not calibrated against any measurements. Stress profiles across the pillars at the marked positions in Figure 5-3 are shown in Figure 5-4 to Figure 5-6 for the three loading conditions.

Table 5-3 FLAC model properties used to compare the effects of boundary conditions

C_0 (MPa)	ϕ_0	ϕ_{res}	ε_{pr} (m ε)	ψ_0	ψ_{res}
30	40	40	100	10	10

C_0 = cohesion

ε_{pr} = residual plastic shear strain

ϕ_0 = internal friction angle at peak load

ψ_0 = dilation angle at peak load

ϕ_{res} = residual internal friction angle

ψ_{res} = residual dilation angle

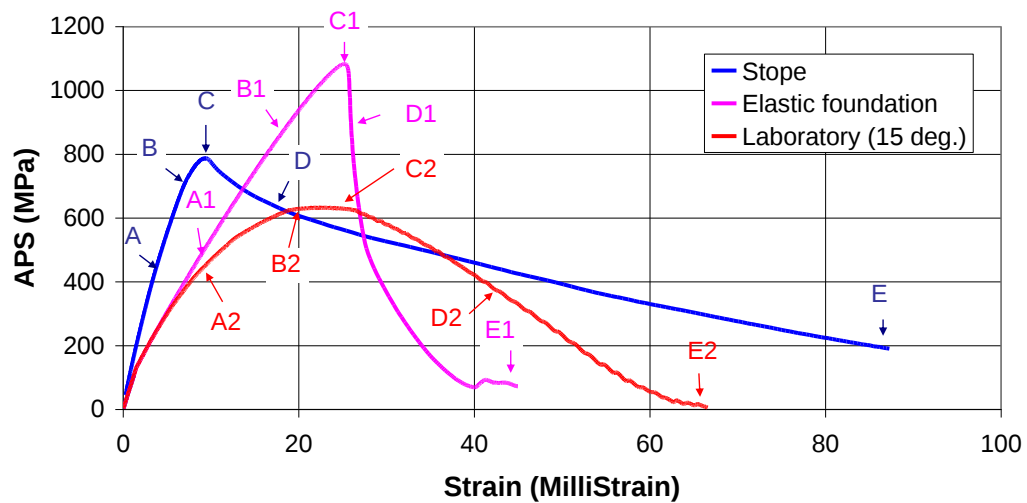


Figure 5-3 FLAC models comparing the effects of draping without foundation damage and no interface (stope), with the same scenario but without draping (elastic foundation), and a scenario with a low friction interface and no draping (laboratory)

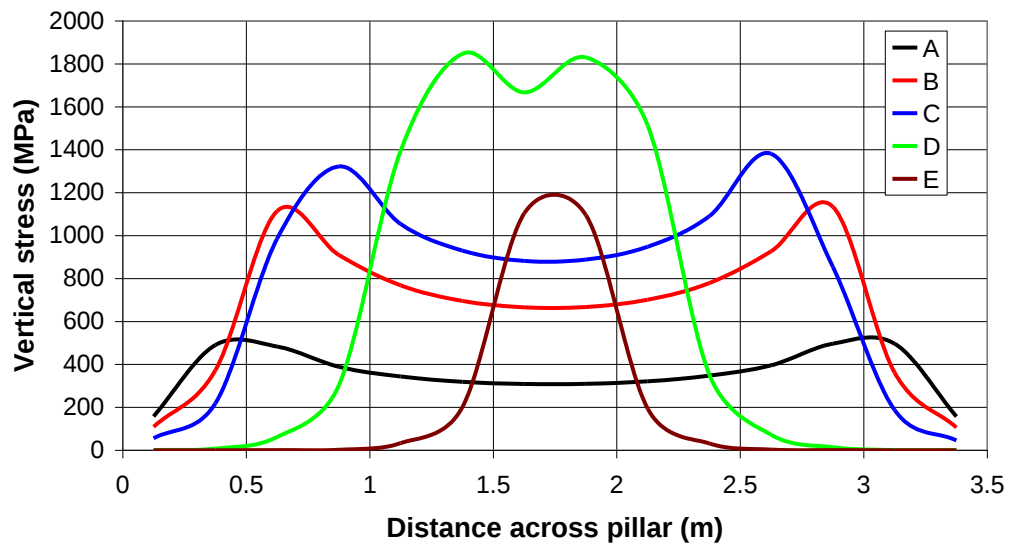


Figure 5-4 Stress profiles across a pillar with no interface between the pillar and rigid foundations, showing the effects of draping. Profiles A – E refer to the positions marked in Figure 5-3

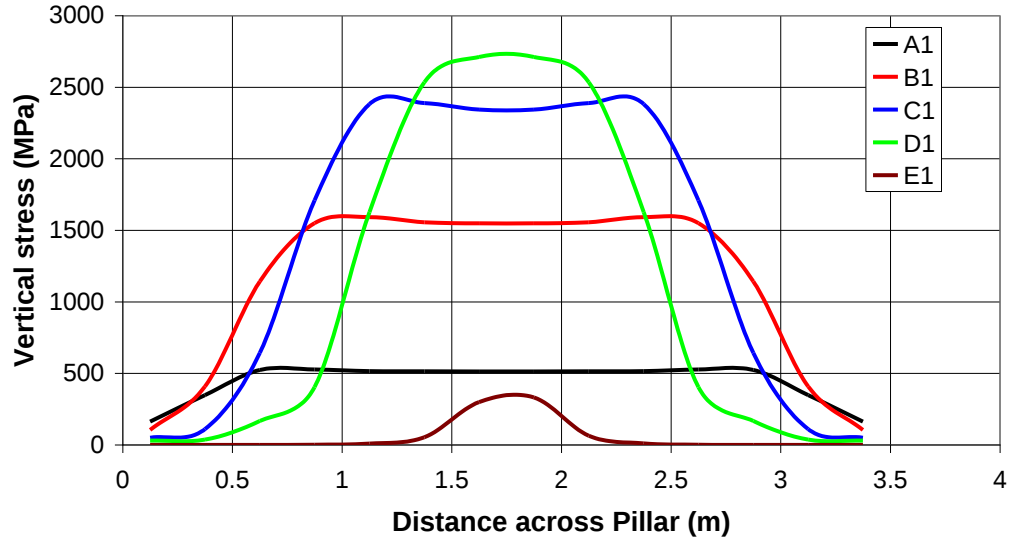


Figure 5-5 Stress profiles across a pillar not affected by draping and no interface between the pillar and rigid foundations. Profiles A1 – E1 refer to the positions marked in Figure 5-3

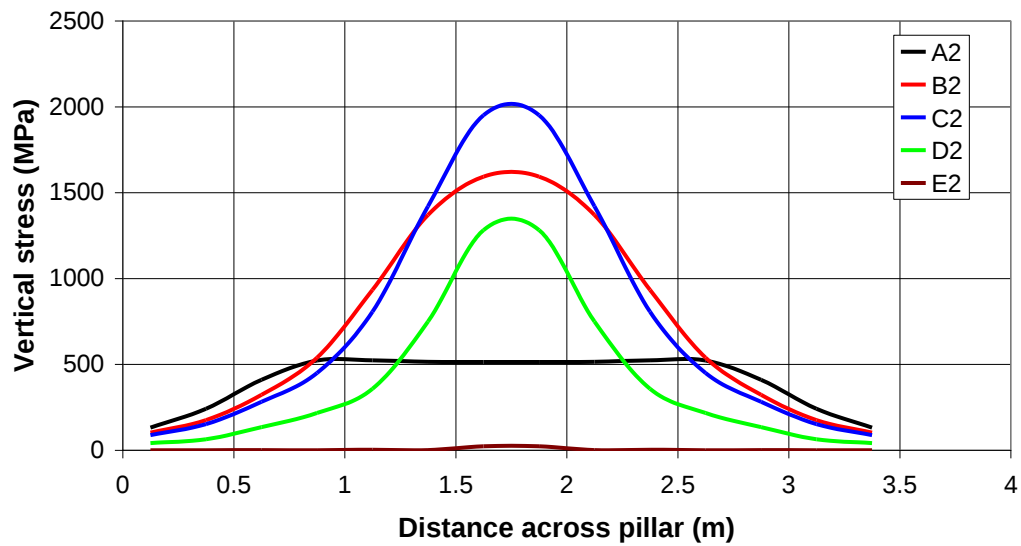


Figure 5-6 Stress profiles across a pillar with an interface of 15° between the rock and rigid foundations, not affected by draping. Typical laboratory scenario. Profiles A2 – D2 refer to the positions marked in Figure 5-3

The draping effect of the hangingwall over the pillar results in high peak stresses at the edge of the pillar before failure (Figure 5-4). Therefore, early failure initiates on the edge and progresses towards the centre. The capacity of the interface to transfer lateral/horizontal stress from the foundation to the pillar results in the inner core of the pillar being confined and thus strengthened. This strengthening effect is illustrated by the comparison between the elastic foundation and laboratory results shown in Figure 5-3. The stope pillar in Figure 5-3 is initially stiffer and subsequently more ductile than either of the other loading environments, again resulting from an early peak stress at the edge of the pillar and the progression of failure towards its centre. Note that the highest peak in Figure 5-4 occurs after pillar failure (Figure 5-3), which is similar to the observation made by Wagner (1980) and is shown in Figure 5-7. In reality, the extent of mining around underground pillars determines the amount of hanging- and footwall drape and hence the severity of this influence.

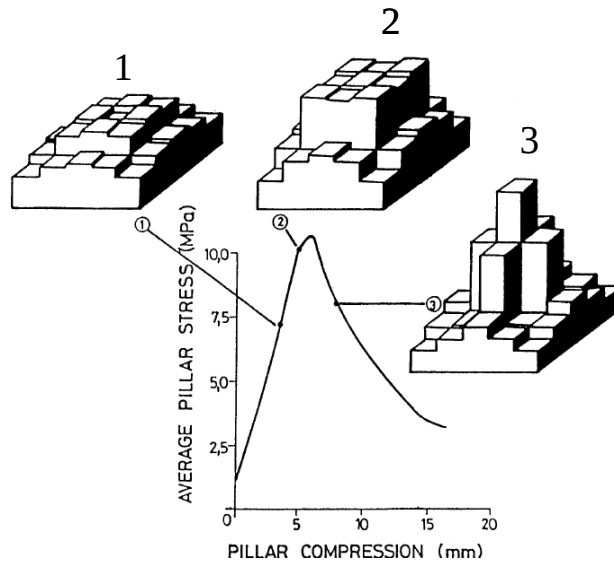


Figure 5-7 Wagner's (1980) in situ tests on coal pillars, showing the stress profile across a pillar for three APS levels (1 = elastic, 2 = yield and 3 = post-failure)

The low frictional effect between the metal platens and the pillar in the test laboratory (York, 1998) results in a lower confinement to the inner core than for the other two loading scenarios and, thus, the laboratory pillar has the lowest peak strength (Figure 5-3). Foundation failure was not studied in these models but the effect of such failure would have reduced the peak pillar strength as shown in the second set of models described below.

The model results seen in Figure 5-3 show the importance of including the foundations with realistic stope geometries as the draping of the hangingwall and footwall contribute to the overall behaviour. Pillar behaviour cannot be simply extrapolated from laboratory tests between metal platens. The modelling described in Section 4.2 shows the importance of allowing failure to progress into the foundations.

5.4 Modelled relationship between w/h ratio and peak strength

5.4.1 Model description

A second set of FLAC models was prepared to determine the peak pillar strength for a wide range of w/h ratios. The input parameters for these models (Figure 5-8) were the same as those used to determine the foundation damage as described in Section 4.2 and to verify the back-analysed pillar strength formulae (Section 5.5.9). Constant-height and constant-width models were run for comparatively ductile and brittle materials. The constant height models consisted of 48 square elements across the height of the pillar, which was kept constant. The constant-width models consisted of 144 elements across the width, and the height of the individual elements was varied – i.e. the elements were rectangular shaped. In all models the stope span was about five times the pillar width (extraction ratio ~ 83%) and the model height was more than eight times the pillar width. The more ductile material was calibrated from underground pillar measurements at the Amandelbult site.

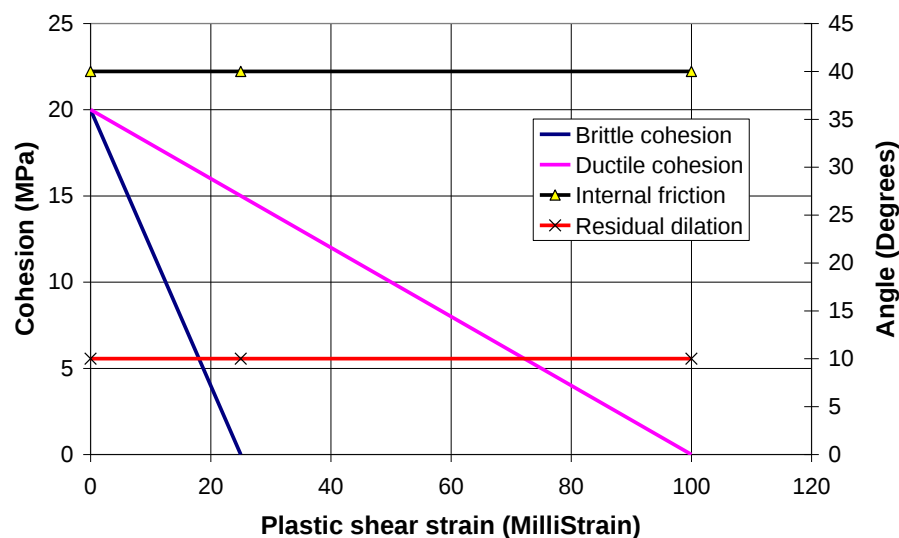


Figure 5-8 *FLAC model properties (with the more ductile model calibrated from underground measurements)*

5.4.2 Model results

The results of the modelling are shown in Figure 5-9. There was no variation in the constant-height- and constant-width model results at low w/h ratios and only the constant width models were conducted at higher w/h ratios. Each result between w/h ratios of four and eight was determined by the sum of results from the constant-height- and constant-width models.

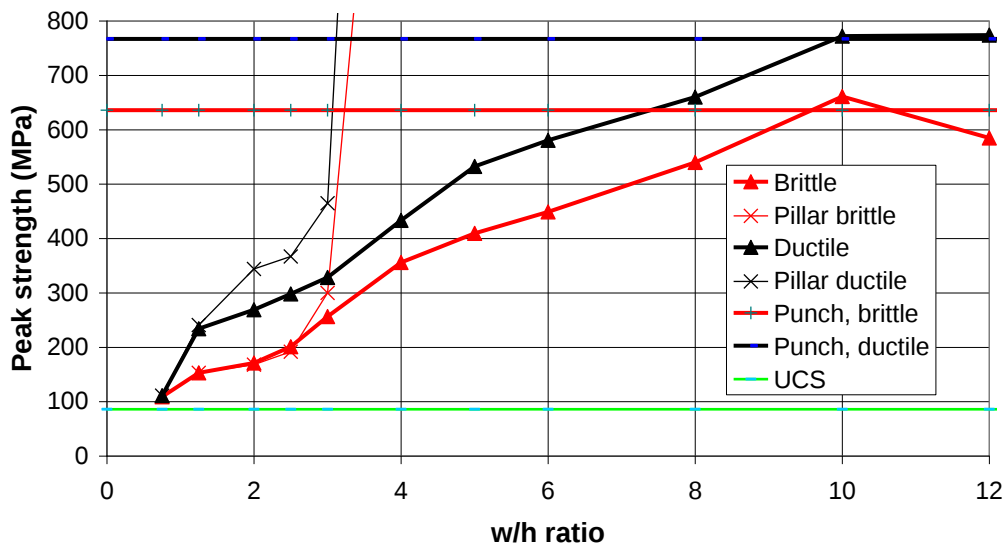


Figure 5-9 Effect of pillar w/h ratio for pillars that are allowed to punch, as well as for pillars that are surrounded by an infinitely strong rock mass. High density mesh and varying brittleness

The graphs labelled "Pillar brittle" and "Pillar ductile" in Figure 5-9 refer to models in which the hanging- and footwall material was not allowed to fail, so that punching was not possible and failure was concentrated in the pillar. Figure 5-9 also shows the pillar UCS, along with the ultimate punch resistance for these relatively brittle and ductile materials. These are labelled "Punch brittle" and "Punch ductile". The modelling showed that the strengthening effects of w/h ratio occurred from a ratio of about 0.6. The higher w/h ratios are associated with a higher aspect ratio of the pillar elements. This change in aspect ratio results from a reduction in the pillar element height. An extreme aspect ratio may explain the reduction in strength at the highest w/h ratio of 12 for the brittle model. It is not immediately obvious why the ductile model is not affected. Despite this numerical artefact, Figure 5-9 shows plausible trends.

As the graphs in Figure 5-9 are based on a UCS of 86 MPa, a change in the value of the UCS would affect the values in the graphs proportionally. The figure shows an approximately linear relationship between strength and w/h ratio up to a ratio of greater than eight. A subsequent suite of models were assembled assuming a residual cohesion of 1 MPa and very slow loading rates. In these models, the parameters for the 'ductile' pillar were used with 144 square elements across the width of the pillar, and the height was varied by changing the number of elements in the pillar height. Regardless of the different cohesion and loading rate, the peak strength of the latter models correlated well with the original models up to a w/h ratio of about six (Figure 5-10). The latter models, however, clearly indicated nonlinear behaviour above a w/h ratio of greater than six with the ultimate pillar strength being reached above a w/h ratio of about 12. This critical w/h ratio at which the ultimate pillar strength is achieved is in contrast to Figure 5-9, where a critical w/h ratio of ten is shown. The original models were probably run at an excessively fast loading rate, which artificially increased the peak strength of the higher w/h ratio pillars.

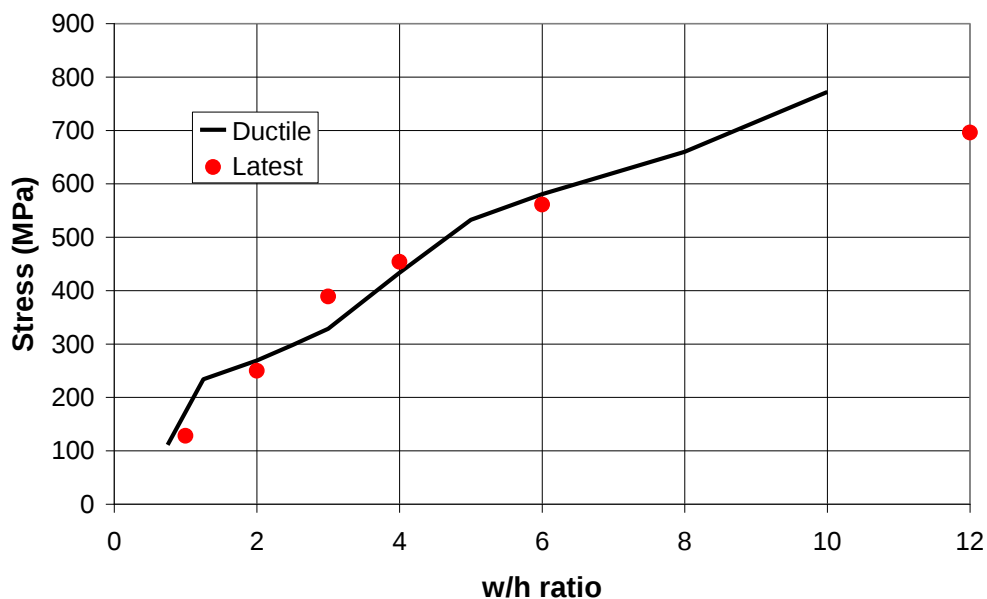


Figure 5-10 FLAC modelling: effects of a very slow loading rate and the inclusion of a 1 MPa residual cohesion (latest) on the 'ductile' models in Figure 5-9

5.4.3 Influence of foundations

The disparity shown in Figure 5-9 between the models with and without elastic foundations shows a change in the mode of pillar failure. This change occurs once the pillar w/h ratio exceeds a certain value. At smaller w/h ratios, the pillars fail by progressively crushing from the edges towards the core, but in the wider pillars additional fracturing of the hanging- and/or footwall rock is initiated. The pillar punching initiated at a stress of about 250 MPa ($\sim 3 \times \text{UCS}$). This punching was associated with different w/h ratios in the different materials. Little or no damage is likely in the foundations below these critical w/h ratios. In the more ductile models, the initial rate of strength increase with w/h ratio appears to have been steeper until the initiation of foundation failure.

The modelling indicated that if the hanging- and footwall material is relatively strong and failure is restricted to the pillar, the so-called “squat” pillar effect occurs at a width-to-height ratio of about three. The graphs labelled “Pillar ductile” and “Pillar brittle” in Figure 5-9 illustrate this effect. The reason why the “squat” effect is not often observed at such small w/h ratios underground is probably because fractures do, in fact, form in the hanging- and/or footwall during loading.

Laboratory experiments on hard rock pillar specimens, loaded between steel platens, have not demonstrated such an extreme exponential relationship between w/h ratio and strength. However, it should be emphasised that the boundary conditions in such laboratory experiments are not representative of in-stope pillars. The interface between the loading platen and the specimen provides limited friction (York, 1998) while the draping effect of the stope is not represented. As a consequence, the laboratory specimens experience far less confinement than the *in situ* pillars and numerical modelling results are probably more representative of actual pillar behaviour.

A more realistic model includes the presence of the hanging- and/or footwall. In such a model the fracturing or damage can expand beyond the pillar itself. This punching phenomenon becomes an important aspect of the failure mechanism of the pillar system, and effectively controls the pillar strength at larger width-to-height ratios. The graphs in Figure 5-9 suggest an approximately linear increase

in pillar strength with an increasing w/h ratio. At relatively large w/h ratios, the punch resistance does, however, reach a maximum at the stress levels indicated in the figure. These levels indicate the ultimate punching resistance of infinitely stiff and strong pillars.

Figure 5-11 suggests that the pillar with a w/h ratio of 2.0 is completely crushed, with limited failure in the footwall. However, the pillar with a w/h ratio of 5.0 shows extensive footwall failure combined with relatively large solid wedges in the core of the pillar. Figure 5-12 and Figure 5-13 indicate that significantly less deformation was shown for the pillar with a w/h ratio of 2 than for the larger w/h ratios, suggesting that the pillar system is softened at the larger w/h ratios. The measured closure is affected, as a consequence.

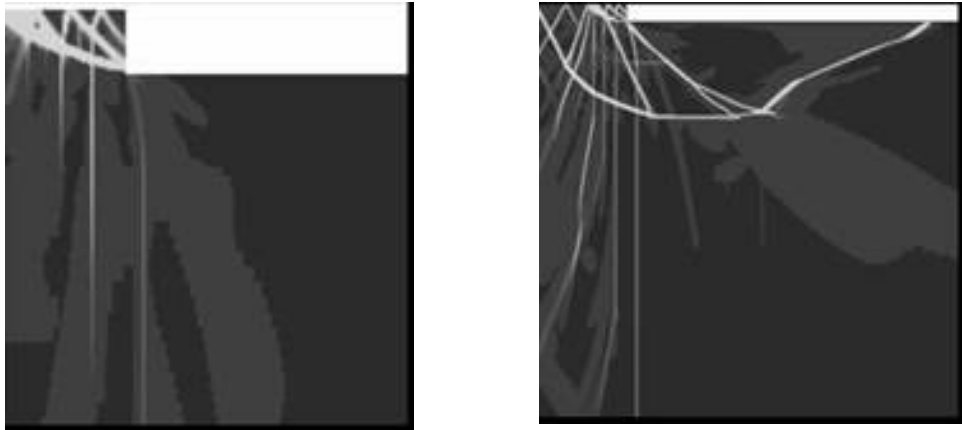


Figure 5-11 Failure distribution, using dense mesh and ductile material; w/h = 2.0 (left) and 5.0 (right) (double symmetry)

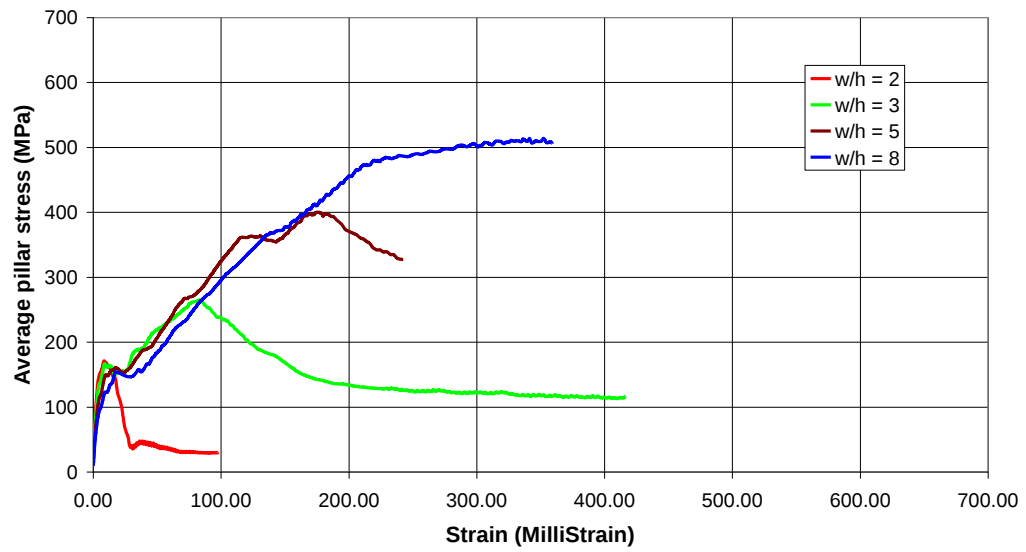


Figure 5-12 Load-deformation relationship; dense grid and most brittle material

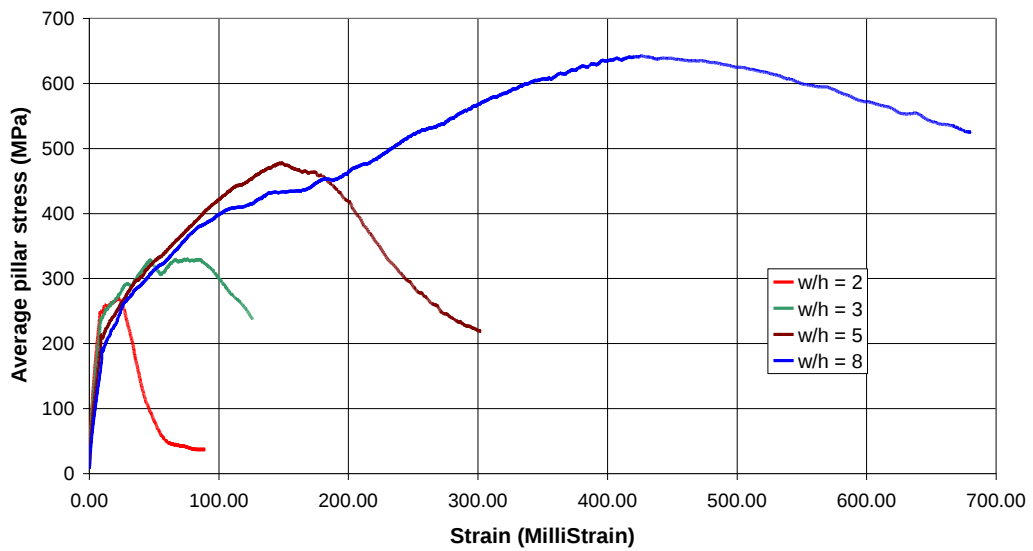


Figure 5-13 Load-deformation relationship; dense grid and least brittle material

The typical Prandtl wedge (Prandtl, 1921) formed in the footwall enables the footwall material to dilate into the stope, thus accommodating the actual pillar deformation and failure. Failure of a pillar system, which includes the adjacent footwall and/or hangingwall rock, involves in essence a combination of three mechanisms. First, there is fracturing and crushing of the pillar itself, which often is reproduced under laboratory conditions with unrealistic boundary conditions. Then there is the fracturing into the surrounding material, the Hertzian crack (Hertz, 1896) and wedge formation. The third mechanism is the horizontal dilation of the foundation, which controls the ultimate resistance against punching. These last two mechanisms have only been investigated to a very limited extent as far as brittle materials are concerned and references are therefore sparse (Cook et al, 1984; Dede, 1997; Özbay and Ryder, 1990; Wagner and Schümann, 1971).

It is, however, clear that the failure of realistic hard rock pillar systems, with the probable exception of very slender pillars, is to a large extent controlled by the fracture and failure processes in the foundation. This was also reported by Lenhardt and Hagan (1990), who observed foundation failure in a pillar at Western Deep Levels Gold Mine. These processes of failure and interactions between the pillar and foundations thus need to be included in any realistic hard rock pillar analysis.

Spencer and York (1999) observed foundation failure in specially prepared laboratory punch tests. These tests included a foundation confined by a metal ring and a rock punch. Both the foundation and the punch in the tests were of the anorthosite rock type, typical of the immediate footwall and pillars at Impala Platinum. After the tests had been conducted, the foundation was cut through its centre vertically to observe the damage. A graph showing the relative depth of the observed damage is shown in Figure 5-14. Unfortunately the confinement of the metal ring on the foundation was not measured.

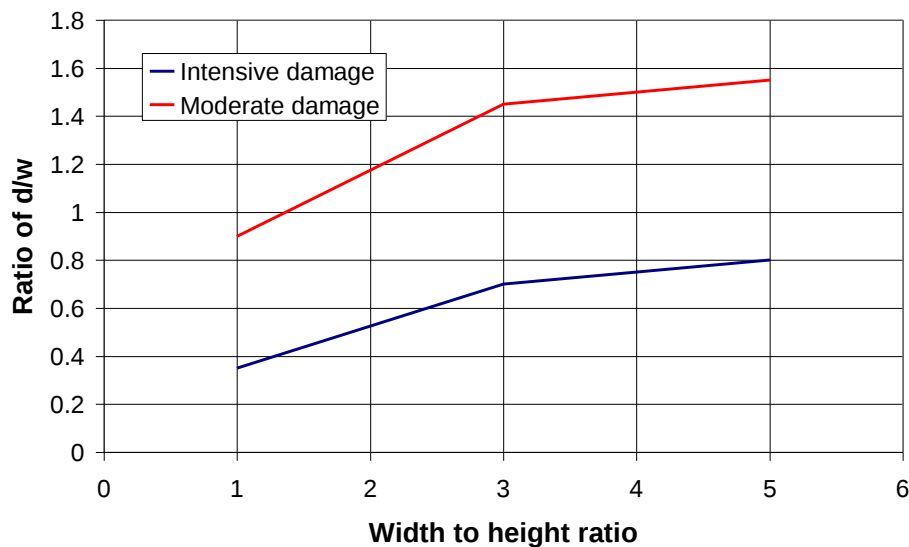


Figure 5-14 Damage below a punch test (after Spencer and York, 1999)

In situations where the pillar material is relatively weak in comparison to the foundations – e.g. a coal pillar between strong sandstone foundations – the foundation damage may be limited and the “squat” effects will therefore be dominant. However, under loading conditions where the hangingwall and footwall have similar geomechanical properties to those of the pillar, foundation damage is inevitable, particularly for wider pillar systems. Further work is needed to establish the conditions necessary for pillars to manifest “squat” effects.

The results of the numerical models clearly show that pillars need to be viewed as a system that incorporates the immediate hanging- and footwall, as well as the pillar itself. With increasing w/h ratio, failure is not contained solely within the pillar, but also expands into the hanging- and/or footwall. The so-called “squat” effect is still present, but it no longer dominates the pillar system behaviour. Increasing pillar strength and pillar load result in increasing damage and failure in the hanging- and/or footwall. At higher w/h ratios, pillars may not necessarily fracture throughout, but the system fails with ensuing load loss. A further conclusion is that little strength benefit is gained by cutting pillars with a w/h ratio greater than about ten.

5.4.4 Influence of material brittleness

In the models, brittleness is controlled by post-failure deformations and grid size. A simple model was built to capture the influence of this post-failure behaviour. The model included a Mohr-Coulomb failure criterion with strain softening. A constant friction angle was assumed to minimise parameter variations and the cohesion was decreased linearly in order to keep the model as simple as possible. In typical uniaxial and triaxial tests, strength is not affected by post-failure behaviour as the material is uniformly stressed and there is little or no confinement of the central core by the surrounding rock. In pillars, failure progresses from the edges towards the core and this process is very sensitive to the post-failure behaviour of the material. The post-failure behaviour of the material controls the load-deformation characteristics of the pillar to a large extent. This important parameter of brittleness is typically ignored in pillar modelling and analysis. Even a simple model can provide some unique insights.

The fact that material brittleness has such a profound effect on system strength can be explained on the basis of the pillar-failure process. Unlike in triaxial tests, where uniform stress conditions prevail prior to specimen failure, pillar failure initiates at the pillar edges and progresses gradually towards the core of the pillar. Edge failure typically starts at a relatively low average pillar stress. Failure progression towards the pillar core is to a large extent controlled by the post-failure behaviour of the previously failed material near the pillar edge. A relatively ductile material would provide more resistance during its post-failure degradation, as it requires more deformation to be completely destroyed. Pillar failure progression will therefore be more restrained in the case of a more ductile material as compared to a more brittle material. This greater restraint implies that an increasing pillar w/h ratio will be associated with a larger rate of strength increase in the case of a relatively ductile material, while the rate of strength increase will be lower in the case of a very brittle material. This rate of strength increase is consistent with the non-punching pillar results shown in Figure 5-9. There was approximately a four-fold difference in the rate of strength increase between the brittle and ductile materials in these models.

The relationship between pillar strength and w/h ratio predicted from the FLAC modelling is also influenced by mesh density. At higher densities, an increase in w/h ratio brings with it less of an increase in predicted strength. This can be explained from the fact that an increased mesh density leads to an increase in effective brittleness. Fracture localisation is enhanced in the case of a denser mesh, which implies that foundation fracturing is more likely to occur in a model with a fine mesh than in a model with a coarse mesh. However, foundation fracturing is not synonymous with foundation failure. Foundation failure is the final stage, in which vertical punching is accommodated by horizontal dilation. It appears that this dilation is induced at a lower resistance level when the element sizes are relatively large. In other words, a reduction in element size (and thus an increase in mesh density) would cause an increased punch resistance. This is in contrast to the effects of mesh density on the crushing of the pillar itself and on foundation fracturing.

In order to obtain a representative material brittleness as well as a correct correlation between pillar failure and rock mass failure, the combination of mesh density and rate of cohesion softening needs to be calibrated properly. Most appropriate, obviously, would be the combination that results in the most accurate estimates of a wide range of pillar strengths. Figure 5-12 and Figure 5-13 show the load-deformation characteristics and failure distributions for various pillar geometries and cohesion softening rates.

5.5 Peak pillar-strength formulae determined from maximum likelihood back-analysis

5.5.1 Introduction

This section deals with the calculation of peak pillar strength, based on the size and geometry of the pillars. In the past, pillar design has been carried out on the basis of experience and strength formulae derived for non-Bushveld hard rock mines. Owing to the uncertainties involved in this methodology, oversize pillars were cut to ensure stability, with the consequential effects on extraction ratio.

Based on visual observations, a database of failed and unfailed pillars was collected from sites where pillar bursts had occurred at the Impala Platinum Mine. The data-collection procedures and site descriptions are provided in Appendix F. The dates of the pillar bursts and the positions of the burst pillars relative to the stope face were known, which meant that stresses could be modelled using MinSim. These stresses were entered into a database together with the measured pillar dimensions. In addition, other pillars in the vicinity that had failed and others that had probably not failed were also entered into the database to provide lower- and upper bounds for the strength analysis. A maximum likelihood back-analysis study was conducted on the data, and strength formulae were developed for the Merensky Reef at Impala Platinum (Figure 5-15). The formulae were primarily used to evaluate the pillar measurements at the Impala site, but good agreement between the formulae and measurements was also observed at other instrumentation sites.

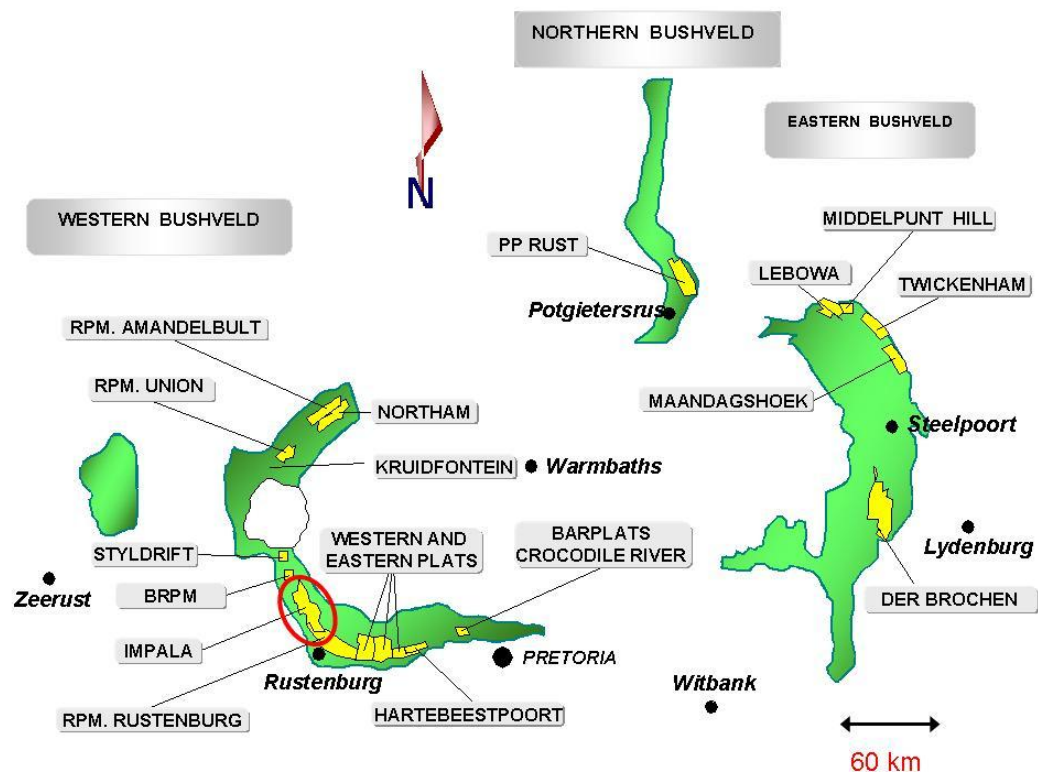


Figure 5-15 Map showing the western, northern and eastern lobes of the Bushveld Complex and highlighting the location of Impala Platinum

5.5.2 Description of evaluated pillars

A total of five stopes on three shafts at Impala was used in the evaluation. The pillars were composite, consisting of pyroxenite and anorthosite with a 1 cm-wide chromitite stringer within the top half of the pillar (Figure 5-16). Generally the pillars consisted of equal parts of pyroxenite and anorthosite. Figure 5-17 and Figure 5-18 show examples of the stress-strain behaviour of the pyroxenite and anorthosite rock types from the instrumented site under uniaxial and triaxial testing conditions. Only one triaxial test with post failure measurements was conducted at each of the three confinements. However, more than 200 UCS tests without post failure were conducted in the anorthosites from this site. Previous testing conducted in the stiff testing machine at the Chamber of Mines Research Organisation (without servo control) showed that the anorthosite rocks generally

failed more violently than the pyroxenites. In addition, it was often possible to conduct a controlled triaxial test into post failure on pyroxenite samples (Figure 5-19), but this was seldom the case for anorthosite samples (Figure 5-20).

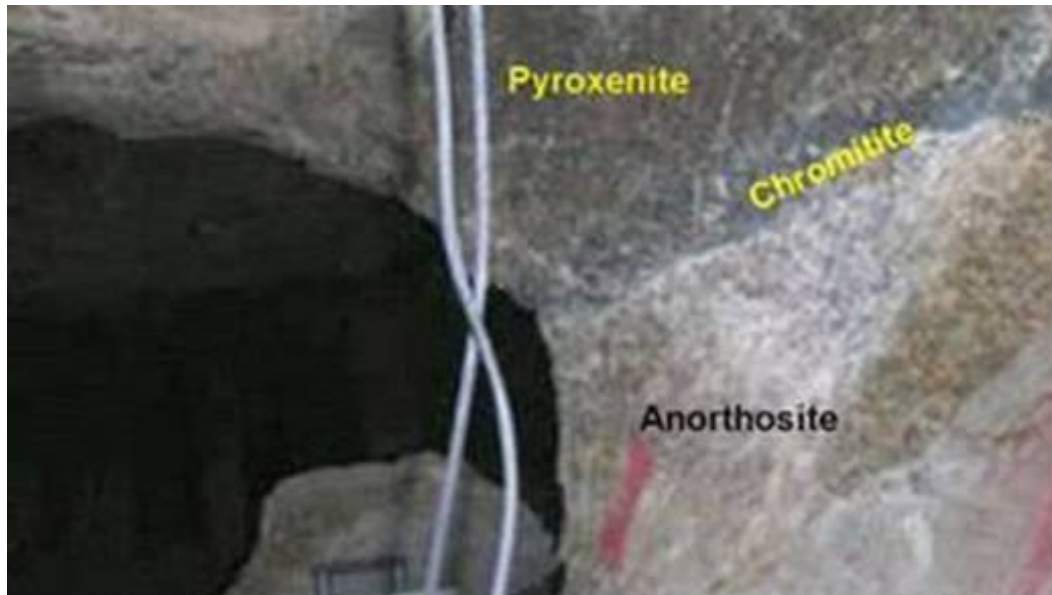


Figure 5-16 A typical composite pillar used in the statistical evaluations

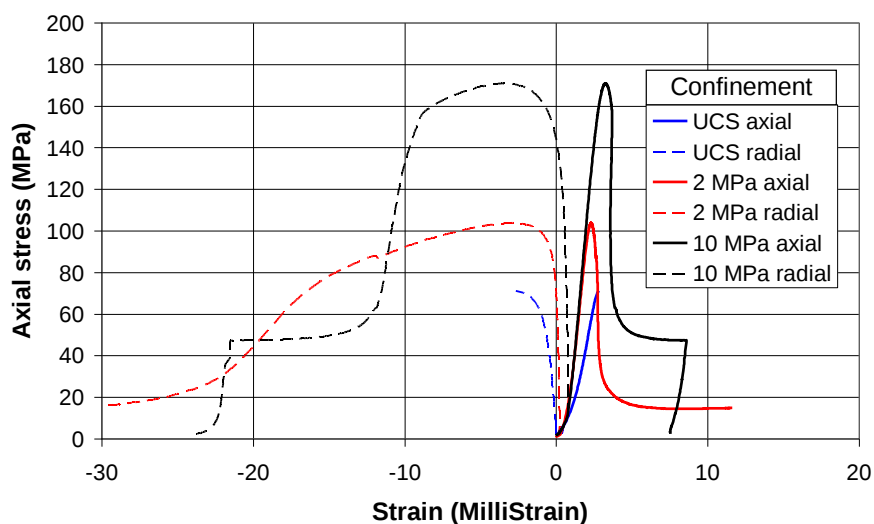


Figure 5-17 Stress-strain behaviour of pillar pyroxenite from the Impala site (same rock type as the immediate hangingwall)

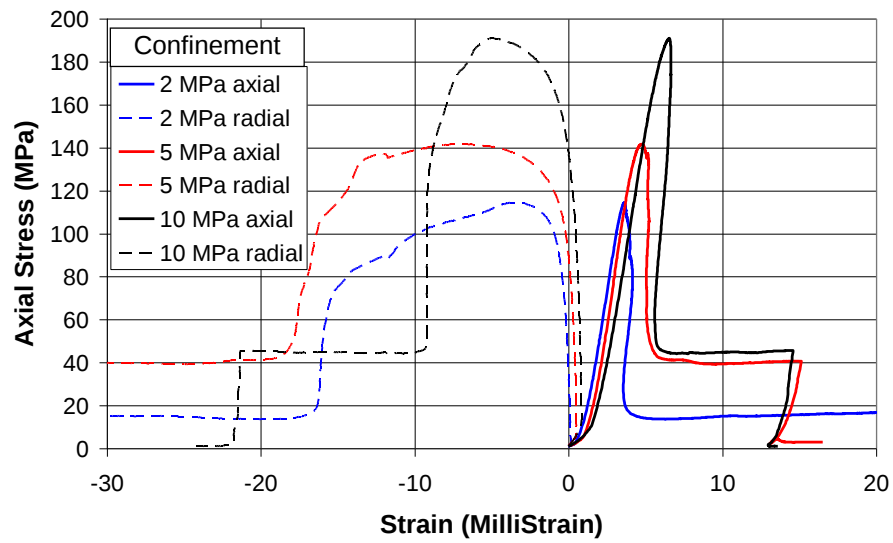


Figure 5-18 Stress-strain behaviour of pillar anorthosite from the Impala site (same rock type as the immediate footwall)

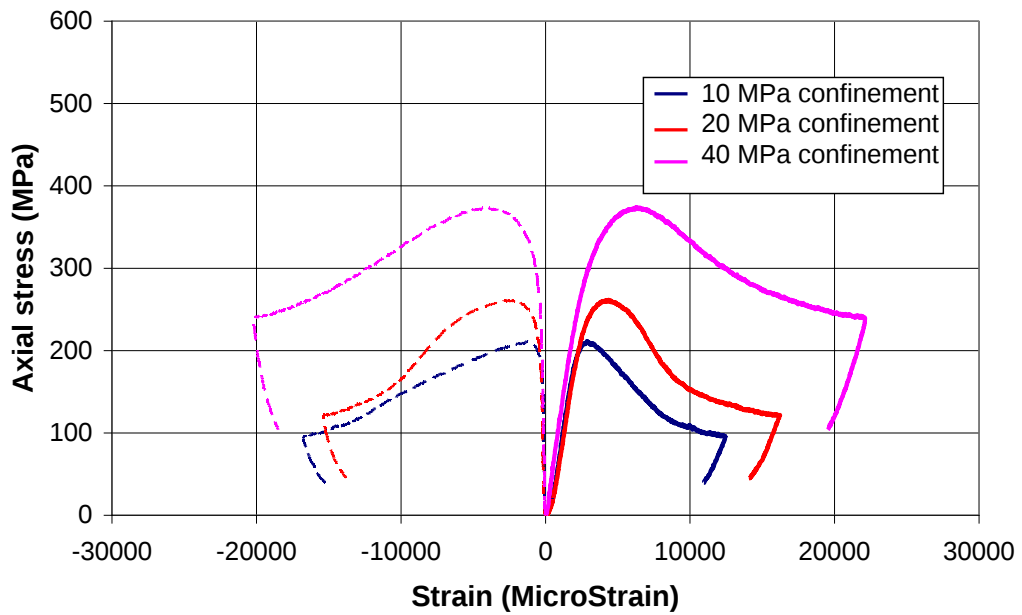


Figure 5-19 Stress-strain behaviour of hangingwall pyroxenite from Amandelbult at shallow depth (400 m below surface)

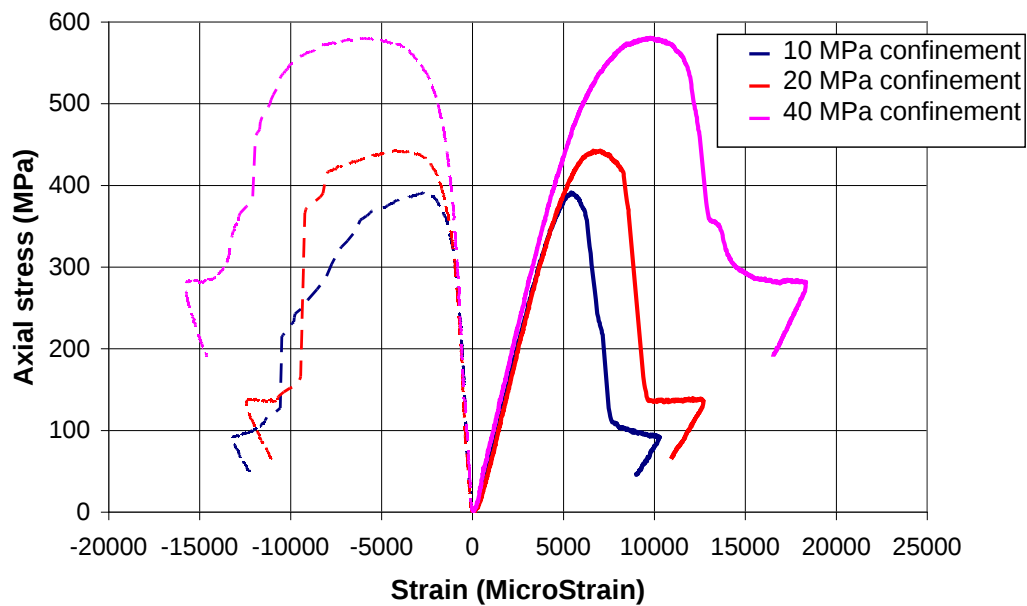


Figure 5-20 Stress-strain behaviour of anorthosite footwall from Amandelbult at shallow depth (400 m below surface)

The geomechanical behaviour of the two rock types shown in Figure 5-17 and Figure 5-18 were surprisingly similar, as tests from the hangingwall and footwall of other shallow-depth Merensky Reef sites often show significant differences. For example, Figure 5-19 and Figure 5-20 show the results of tests conducted on the immediate hangingwall pyroxenite and footwall anorthosite, respectively, from Amandelbult at a depth of 400 m below surface. The elastic constants of the shallow-depth Amandelbult tests were similar to the ‘matrix’ constants described in Chapter 3, suggesting that the behaviour of the rock in the confined areas of the pillar may be better described by the curves shown in Figure 5-19 and Figure 5-20. Chapter 3 shows that nonlinear behaviour occurred in the vertical direction at the Impala site when the vertical stress drops below 10 MPa. As the virgin horizontal stress is higher than in the vertical direction, the nonlinear behaviour threshold may also be higher in this direction. The similar behaviour shown by the materials in Figure 5-17 and Figure 5-18 is probably due to the overwhelming influence of the microfracturing, which occurred when the rock was destressed. Areas in the pillar core where the confinement was sufficient to prevent the development of microfracturing probably experienced complex stress interactions during loading. The average UCS values for anorthosite and pyroxenite from the Impala site are 105 MPa and 80 MPa, respectively. However, all these samples

were affected by microfracturing, which also reduced the rock strength. Areas in the core of the pillar that were potentially unaffected by the microfracturing may have been stronger than the rock on the edge of the pillar. This difference in strength is particularly evident in the anorthosite results in Figure 5-18 and Figure 5-20. The Type II post-failure behaviour (Wawersik, 1968) shown by the anorthosites in Figure 5-18 highlights the brittleness of this rock type and the vulnerability of these pillars to bursting. This behaviour is not observed in the tests shown in Figure 5-20 because the test machine used in the evaluation was not servo-controlled. However, the comparatively higher brittleness of the anorthosites shown in Figure 5-20 is apparent if compared to the post-failure behaviour of the pyroxenites shown in Figure 5-19.

5.5.3 Pillar characterisation

The *in situ* dimensions of the evaluated pillars were directly measured and the presence/absence of sidings adjacent to pillars was noted. Pillar condition was documented according to the following scale of condition codes (CC):

- 5: Pillar heavily damaged, date/geometry at failure not accurately known
- 4: Pillar presumed failed, date/geometry at failure not accurately known
- 3: Pillar definitely failed (or burst), date/geometry at failure known
- 2: Pillar sidewalls visibly fractured/scaled, date/geometry known
- 1: Pillar sidewall scaling barely visible, date/geometry known
- 0: Pillar with no visible damage, date/geometry known.

Pillars with condition code 3 are the most directly relevant for back analysing strength parameters, although pillars with other codes give confirmatory information.

5.5.4 Database description

Descriptions of the sites are provided in Appendix F. The database consisted of 179 pillars, of which 109 represented stresses at some value below the peak strength (CC = 2), 62 provided some stress higher than the strength (CC = 4), and there were eight pillars where the stress condition at failure could be

calculated ($CC = 3$). Pillar effective widths (w_e) account for rectangular pillars, taking cognisance of pillar length (L) according to the widely used ‘perimeter rule’, described by Wagner (1974) and provided in Equation 5-4. Similarly, pillar height was corrected to allow for the presence of gullies unprotected by sidings (h_e), using Equation 5-5. As a typical example, the effective height (in terms of expected strength) of a siding-less pillar with a gully depth of 2.0 m increases from 1.2 m to about 1.6 m. The correction is based on numerical modelling performed by Roberts et al (2002). Further research is, however, required to better establish the effects of gullies and sidings on pillar behaviour.

$$w_e \approx 2 w L / (w + L) \quad 5-4$$

$$h_e \approx [1 + 0.2692(w/h)^{0.08}] h \quad 5-5$$

where w and h are the pillar width and mining height respectively.

The majority of the pillar w_e/h_e ratios ranged between 2 and 6, with the largest proportion being between 2.5 and 3.5 (Figure 5-21). Very little data was available for w_e/h_e below 1.5 or greater than 6.

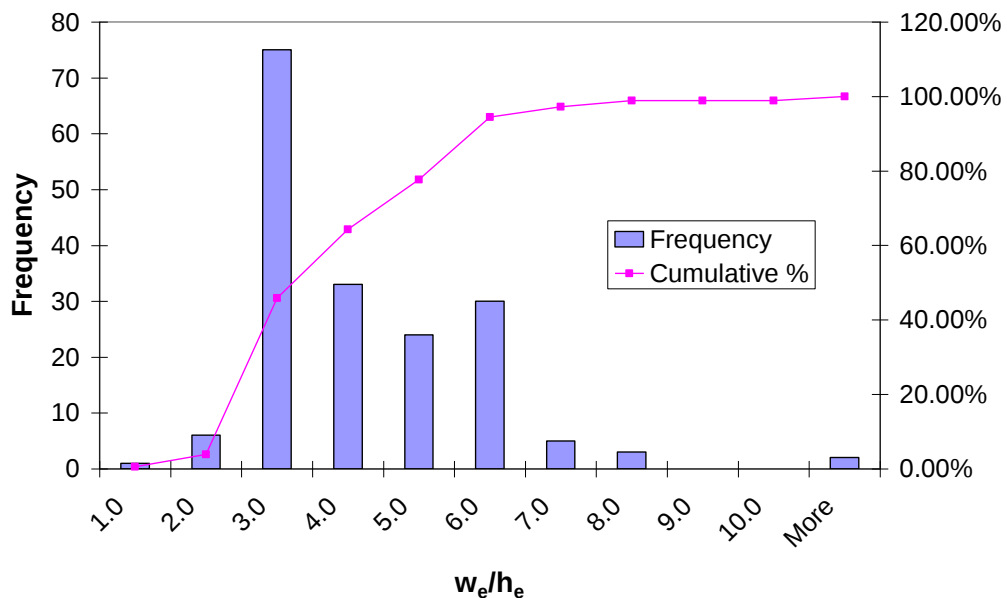


Figure 5-21 Distribution of pillar w_e/h_e in the database

The database included a wide range of pillar lengths (Figure 5-22) and widths (Figure 5-23) but the heights fell into a limited range of between 1.2 m and 2 m (Figure 5-24).

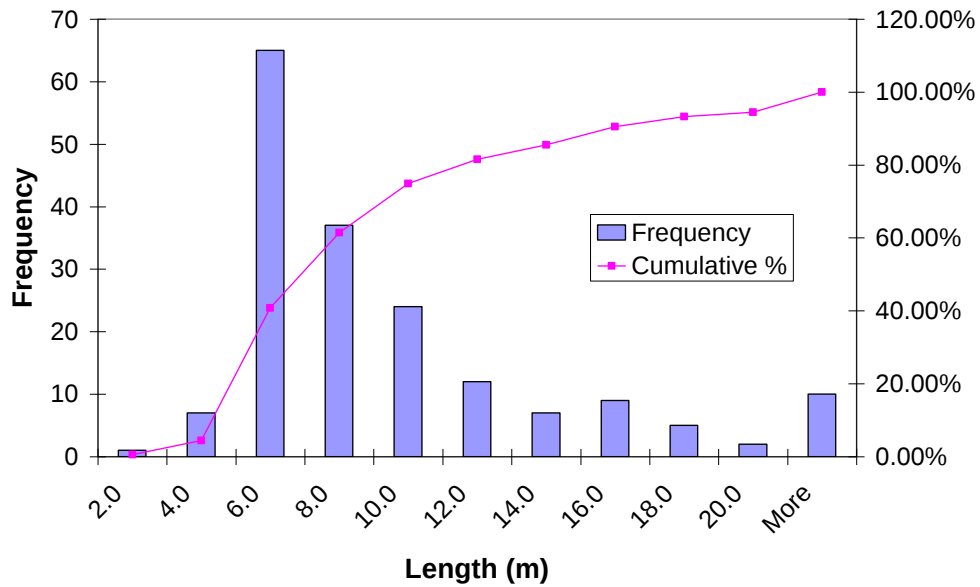


Figure 5-22 Distribution of pillar lengths in the database

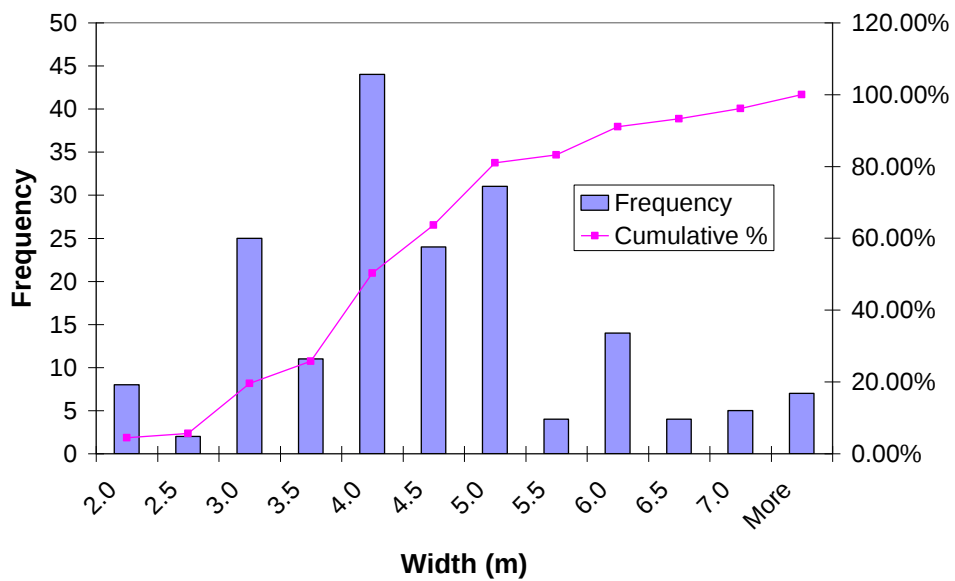


Figure 5-23 Distribution of pillar widths in the database

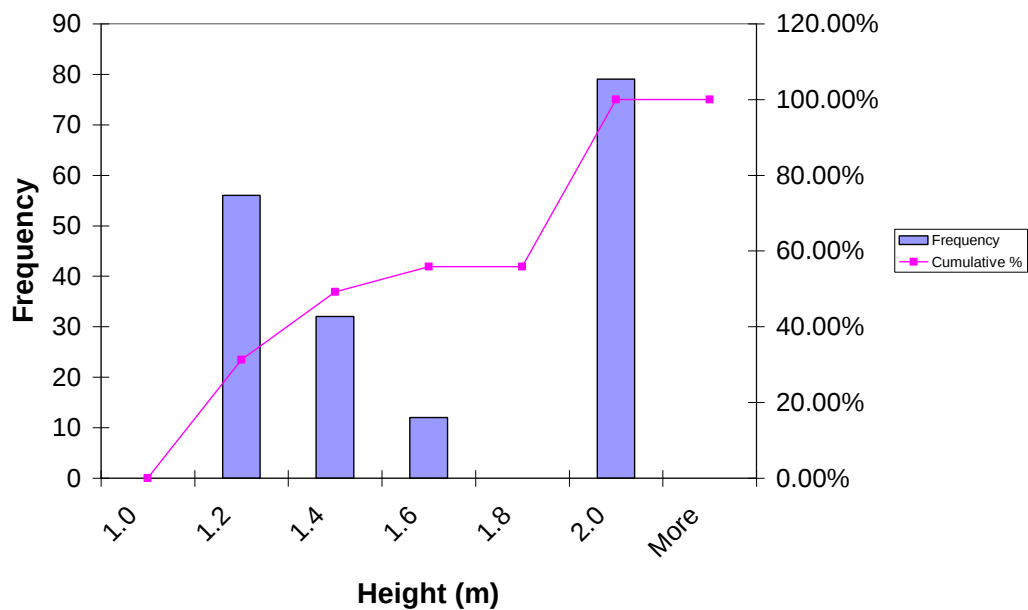


Figure 5-24 Distribution of pillar heights in the database

5.5.5 Strength parameter estimation

The strength of a pillar may be assumed to be a function of its known *physical* characteristics (including width, height, length), and certain unknown *parameters* (e.g. Salamon and Munro, 1967: K , α , β values). The maximum likelihood analysis was used to estimate a ‘best fit’ for these parameters. This type of statistical back-analysis accounts for the many variables that contribute to pillar strength without necessarily needing to understand the failure mechanisms. The process involved the evaluation of a database of APS values determined by MinSim (COMRO, 1981) or MINF (Spottiswoode and Milev, 2002), where the ‘condition’, i.e. ‘intact’ or ‘failed’, was known.

Following the approach of Salamon and Munro (1967), the safety factor (SF) of each pillar was defined by:

$$SF = \text{Strength} / \text{APS}$$

5-6

A probabilistic *distribution* of SFs governs the condition of pillars, in the sense that a pillar with $SF > 1$ is likely to be intact, while one with $SF < 1$ is likely to have failed. A lognormal distribution was assumed for the SFs, having a log mean of zero and standard deviation of s . With this formulation, physically meaningless negative SFs are disbarred, and reciprocal symmetry pertains; e.g. a pillar having $SF = 0.5$ is about as likely to have failed as one with $SF = 2$ to have *not* failed. The logarithmic standard deviation was assumed to account for *all* uncertainties in the system of pillars – e.g. mis-measurement of widths, mis-estimation of pillar APS values, real geotechnical variations in pillar properties, etc. For historical reasons, logarithms to base 10 are used in the lognormal distribution, and to interpret s , $10^{\pm s}$ needs to be evaluated in relation to unity; s is a parameter the numerical value of which has to be estimated along with the unknown parameters governing the strength of the pillars in a given observed set.

A ‘likelihood function’ (L_i) of the probability of the pillars exhibiting their stipulated condition (‘intact’ or ‘failed’) was set up. To avoid multiplications, the logarithm (base e) of L_i was used so that the function F was defined as in Equation 5-7 (Ryder et al, 2005):

$$F = \ln L_i = \sum \ln(\text{prob. of intact cases}) + \sum \ln(\text{prob. of failed cases}) \quad \mathbf{5-7}$$

The probability of an intact case (condition codes $CC = 0, 1$, or 2 in this study) was given by $\Phi(\log SF)$ where Φ is the cumulative normal distribution. Such cases biased the derived best parameter fits so that their SFs were as large as possible.

The probability of a failed case in which the APS value was the estimated load at which *failure actually occurred* (the situation in Salamon and Munro’s (1967) back-analysis, and $CC = 3$ in the present study) was given by $\phi(\log SF)/SF$, where ϕ is the normal probability density function. These cases strongly biased the best-fit parameters so that their SFs were more-or-less symmetrically disposed about unity. Note that this scenario corresponds exactly to the formulation presented in Salamon and Munro (1967).

The probability of a failed case where the APS was merely an upper bound and failure probably took place earlier at some lower APS value (the situation in many

of the back-analysis scenarios, CC = 4 or 5) was expressed by the function $(1 - \Phi(\log SF))$. This is analogous to the treatment of intact cases, and biased the best-fit parameters so that the SFs were as small as possible. The fit is weaker than for the situation where APS values reflect actual strengths at which failure occurred.

Two pillar strength formulae were back fit, using the maximum likelihood evaluation:

- linear formula (Bieniawski and Van Heerden, 1975); and
- power formula (Salamon and Munro, 1967).

5.5.6 Linear pillar strength formula

The linear equation for pillar strength (PS), in its original form (Equation 5-8), assumes square pillars. Ryder et al (2005) modified the equation to obtain an explicit estimate of length (L) strengthening (Equation 5-9), partially based on expected strengthening under plane strain conditions (Ryder and Özbay, 1990).

$$PS = K_i \left[b + (1 - b) \frac{w}{h_e} \right] \quad 5-8$$

$$PS = K_i \left[\frac{1 + a}{1 + aw/L} \right] \left[b + (1 - b) \frac{w}{h_e} \right] \quad 5-9$$

The back-fit values for Equation 5-9 are provided in Table 5-4.

Table 5-4 Back fit values for Equation 5-9

Parameter	Value
K_i (<i>In situ</i> cube strength)	136 MPa
a (Length parameter)	0.27
b (Linear w/h_e parameter)	0.59
s	0.073

The 'a' parameter predicts a 27% increase in the strength of a rib compared to the strength of a square pillar, which is similar to the strengthening effects suggested by Ryder and Özbay (1990) and Roberts et al (2002) (~30%). The 'b' parameter is similar to values obtained by Bieniawski and van Heerden (1975) for large *in situ* South African coal specimens ($b = 0.64$). The predicted *in situ* cube strength (K_i) appears slightly high as the laboratory UCS determined from anorthosite cylinders, with w/h ratios of 0.3, was only about 90 MPa at the Impala site. However, these tests were affected by microfracturing and the rock mass in and around the pillars was not. A more realistic strength for unfractured anorthosite at Impala Platinum Mines is probably about 170 MPa (Figure 5-25).

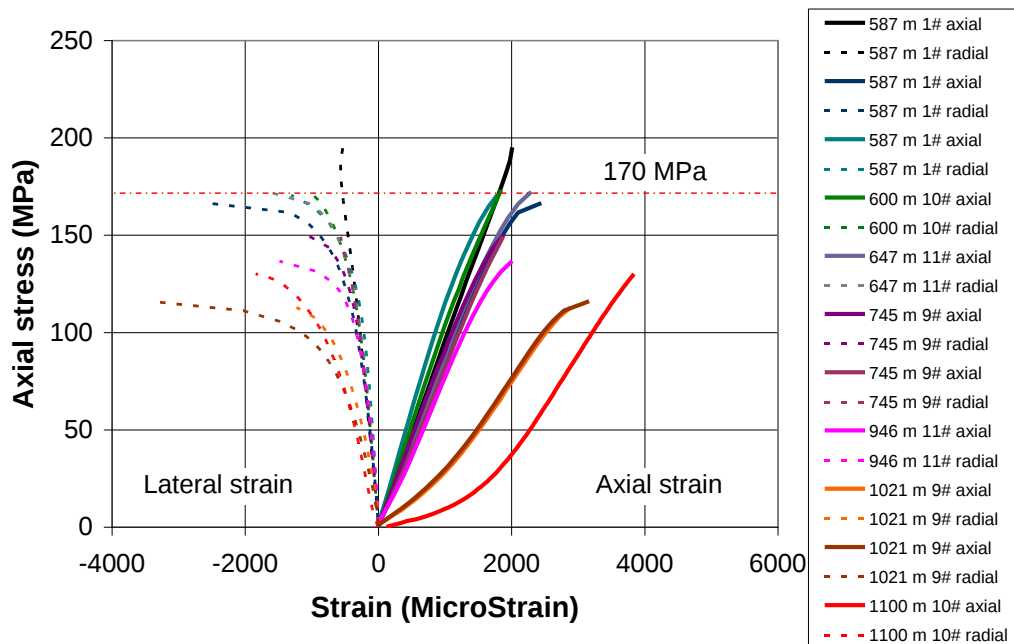


Figure 5-25 Anorthosite UCS tests from a range of depths below surface at Impala Platinum Mine, showing an average strength of 170 MPa for the unfractured rock

A comparison between the modelled APS and calculated strength values is shown in Figure 5-26. The figure shows a good separation between failed and unfailed pillars, with a correspondingly low evaluated standard deviation 's'. The good fit is particularly evident from the CC = 3 cases in which pillar failure strength was obtained with confidence.

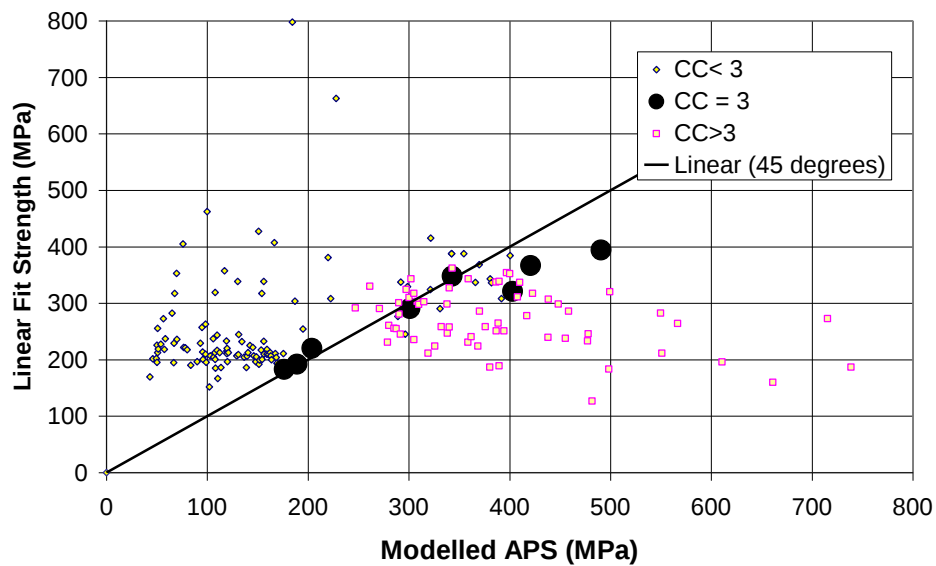


Figure 5-26 Plot of failed and unfailed pillars with the linear formula compared to the modelled strengths (Equation 5-9)

As an illustrative example of the use of Equation 5-9, a pillar with dimensions ($L \times w/h_e$) of 6 m x 3 m / 1.2 m provides an estimated strength of 246 MPa.

The linear equation can also be used assuming the perimeter rule for rectangular pillars (w_e) as determined by Wagner (1974) and shown in Equation 5-4. Equation 5-8 can thus be rewritten as:

$$PS = K_i \left[b + (1 - b) \frac{w_e}{h_e} \right] \quad 5-10$$

Only two parameters are required in Equation 5-10 and the back-fit values are provided in Table 5-5.

Table 5-5 Back-fit values for Equation 5-10

Parameter	Value
K_i (<i>In situ</i> cube strength)	147 MPa
b (Linear w/h_e parameter)	0.70
s	0.075

The results from the Wagner (1974) analysis show a higher *in situ* cube strength (K_i) than the already high value indicated in Table 5-4. The standard deviation 's' was also slightly higher than the Ryder et al (2005) back-fit analysis, which suggests a worse fit than that analysis. However, the 'b' value was similar to widely reported values (0.78) for materials including coal, norite and sandstone (Ryder and Özbay, 1990).

A comparison between the modelled APS and calculated strength values from the Wagner (1974) analysis is shown in Figure 5-27. The figure also shows a good separation between failed and unfailed pillars, but the strength prediction for the CC3 pillars is not as good as in Figure 5-26, particularly for the higher w/h ratios.

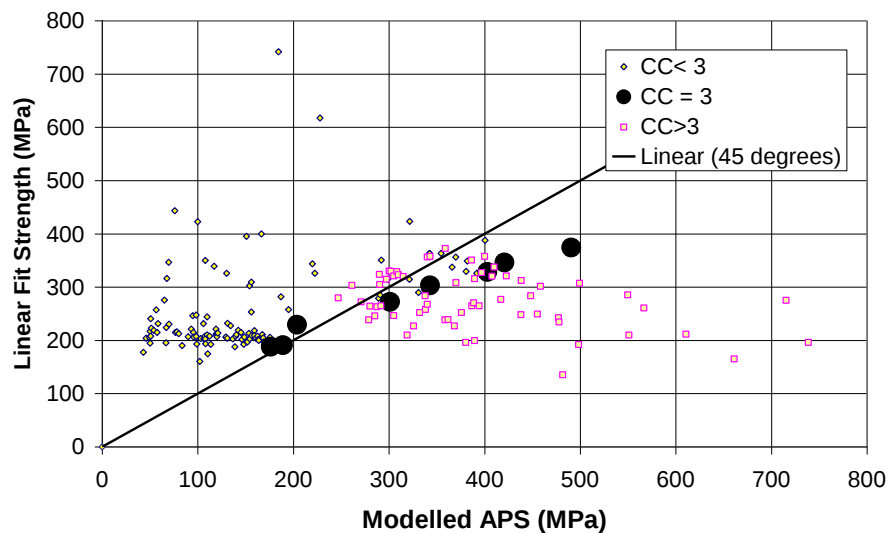


Figure 5-27 Plot of failed and unfailed pillars with the linear formula and perimeter rule compared to the modelled strengths (Equation 5-10)

5.5.7 Power formula for pillar strength

The results of the standard power formula back fit (Equation 5-11) are shown in Table 5-6. The length strengthening effects are implicit in the use of the perimeter-rule (Wagner, 1974) and the effective width is shown as: w_e in Equation 5-11.

$$PS = Kw_e^\alpha h_e^\beta \quad 5-11$$

Table 5-6 Back-fit values for Equation 5-11

Parameter	Value
K	86 MPa
α (Effective width parameter)	0.76
β (Effective height parameter)	-0.36
s	0.080

The α and β values shown in Table 5-6 differ significantly from the Salamon and Munro (1967) back-fit analysis for South African coalmines ($\alpha = 0.46$, $\beta = -0.66$). The Hedley and Grant (1972) analysis ($\alpha = 0.50$, $\beta = -0.75$) was also much different to the results of the current investigation. However, the β value in Table 5-6 was determined from a relatively small range of heights (Figure 5-24) and may, therefore, be unreliable. The estimated strength using the power formula for a pillar with dimensions 6 m x 3 m ($w_e = 4$ m) and a height of 1.2 m is 231 MPa, slightly lower than that determined by Equation 5-9.

The modelled APS and calculated strength values are compared in Figure 5-28. Again a good separation between failed and unfailed pillars was indicated, but the standard deviation was slightly higher than that for the analyses shown in Table 5-4.

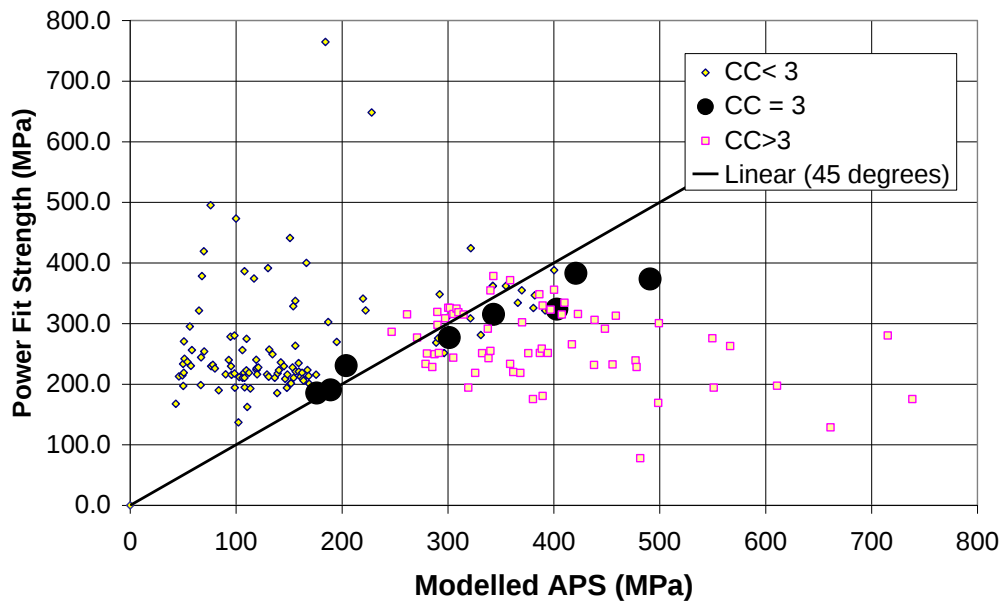


Figure 5-28 Plot of failed and unfailed pillars with the power pillar-strength formula compared to the modelled strengths (Equation 5-11)

5.5.8 Comparison of formulae

The initial investigations described above suggest a good correlation between calculated and actual strengths for both the linear and power formulae. However, the linear formula provides a slightly smaller standard deviation and, therefore, better results for the whole database. This linear relationship between strength and w/h is also supported if just the eight CC3 pillars are considered, as in Figure 5-29 and Figure 5-30.

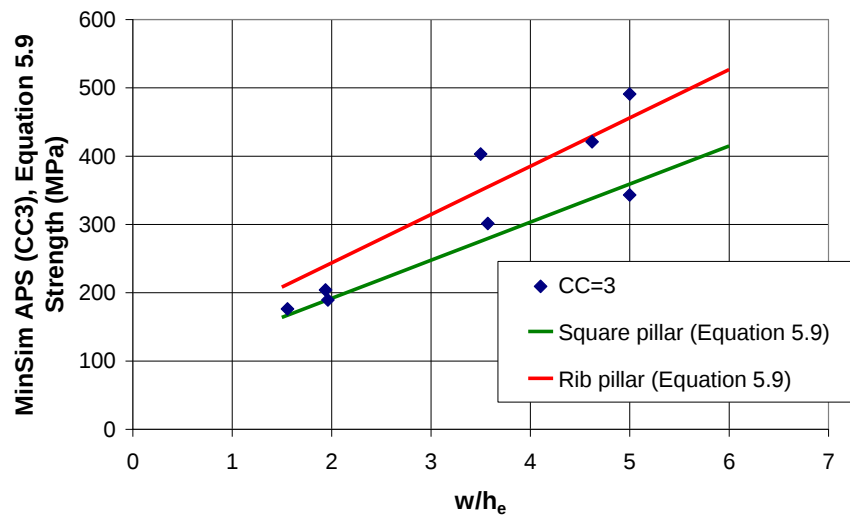


Figure 5-29 CC3 pillar strengths (MinSim) and Equation 5-9 for square and rib pillars as a function of w/h_e

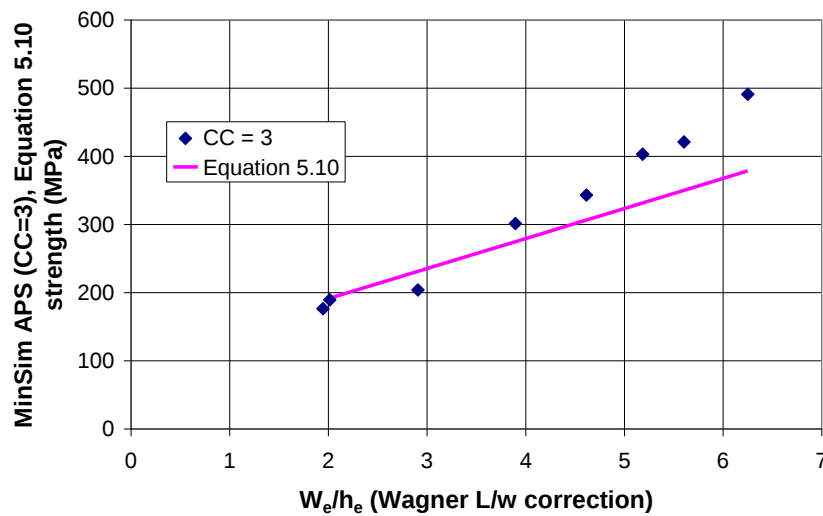


Figure 5-30 CC3 pillar strengths (MinSim) and Equation 5-10 as a function of w_e/h_e

The results of the CC3 pillars show a similar relationship between pillar strength and w/h ratio as the rest of the database does. Figure 5-29 matches the CC3 values better than Figure 5-30 does. Theoretically, the CC3 data points should be located between the extremes of the square and rib pillars in Figure 5-29 and on the back-analysed strength line in Figure 5-30. The slope of a CC3 linear regression line is, however, steeper than the slope of the back-analysed strength

equations. These results suggest that the database may slightly underestimate the pillar strength at higher w/h. However, there were only eight CC3 pillars and their statistical relevance might be limited. It is interesting that the back analyses suggest an approximately linear increase in strength even above w/h ratios of five. Squat-pillar theory (Madden, 1991) suggests an exponential increase in strength at a w/h of greater than five. This theory, therefore, does not appear to apply to the pillars in the database; the reason, most probably, is that foundation-failure effects play a large role in determining the system strength of underground hard rock “squat” pillars (Section 5.4). Numerical modelling shows that there is no strength increase above a w/h ratio of ten as the foundation dominates the behaviour. Thus, at w/h ratios that are greater than in the database, the linear law no longer applies. The modelling in Section 5.4 suggests that this could be for w/h ratios above about eight.

Figure 5-31 compares the strengthening effects of the Ryder et al (2005) and Wagner (1974) correction for length. The linear equation that includes the Wagner (1974) correction (Equation 5-10) is influenced by the w/h ratio whereas the Ryder et al (2005) adjustment is independent of this ratio. The comparison in Figure 5-31 covers the range of CC3 widths in the database.

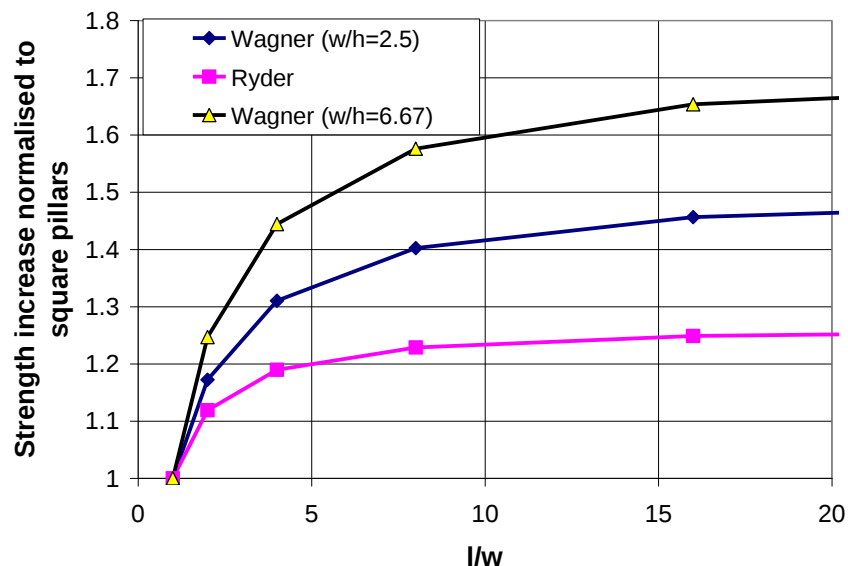


Figure 5-31 Comparison between the strengthening effects of pillar length for the Wagner (1974) (5-10) and Ryder et al (2005) (5-9) equations

The Wagner (1974) width correction for pillar length (Figure 5-30) provides a better correlation coefficient for linear regression analysis of the CC3 pillars than the Ryder et al (2005) evaluation shown in Figure 5-29, but only eight pillars were used in the assessment. In addition, the Wagner (1974) length-strengthening effect for pillars with $w = 3$ m and $h = 1.2$ m, when applied to the linear equation with the constants in Table 5-5 (Figure 5-31), provides similar results to Stavropoulou's (1982) findings – i.e. that a conventionally tested uniaxial sandstone specimen is 45% weaker than a specimen tested under plane strain conditions. However, the Wagner (1974) correction is dependent on pillar w and h (3 m and 8 m curves in Figure 5-31), probably providing unrealistically high factors for the larger pillars in the database. The worse fit of the Ryder et al (2005) regression analysis (Figure 5-29) compared to the Wagner (1974) evaluation (Figure 5-30), and the larger influence of length on strength increase predicted by Stavropoulou (1982), suggests that the Ryder et al (2005) correction for length may be slightly conservative. The conservative nature of the Ryder et al (2005) correction for length, the smaller standard deviation shown in Table 5-4, and the better evaluation of larger pillars by Equation 5-9 suggests that this equation with the parameters in Table 5-4 is preferred over Equation 5-10 for the design of stable pillars.

The value for s in Table 5-4 was used to determine a range of safety factors. These safety factors have been plotted as a function of probability of stability in Figure 5-32 and may be used when designing stability pillars with similar geomechanical and geotechnical conditions to the pillars in the database. A safety factor of 1.7 will provide a probability of stability of 99.9%, on the basis of the limited data in the database.

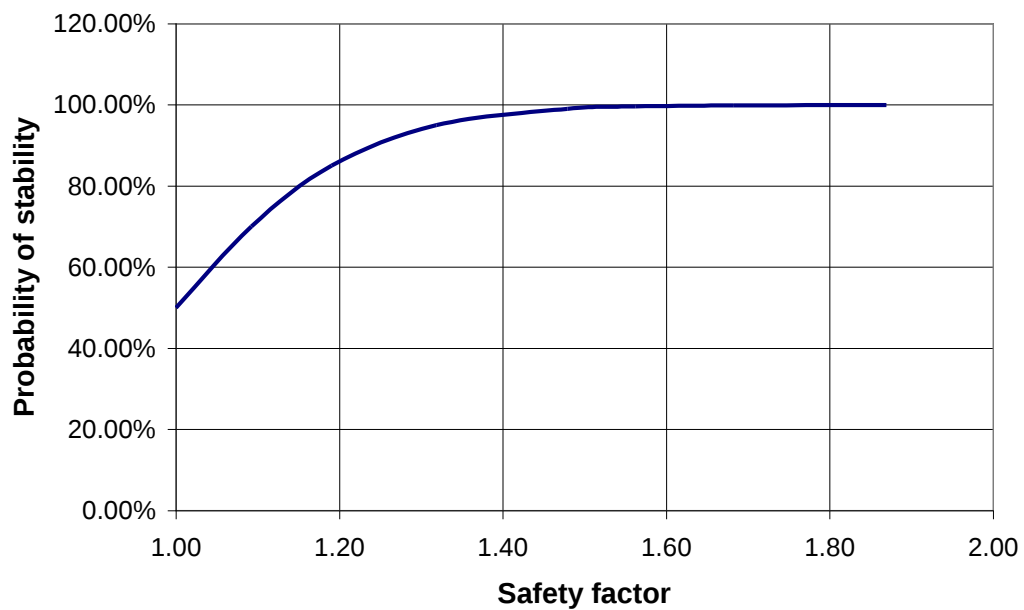


Figure 5-32 Safety factor for the Impala Merensky pillars as a function of probability of stability, based on the linear back-fit analysis ($\log s = 0.073$)

The database consisted of a mixture of pillars with and without sidings, and the siding depths and the heights of gullies also varied. The effect of these parameters on pillar strength was applied using unproven theory and needs to be investigated further.

5.5.9 Formula verification

The models described in Section 5.3 were used to evaluate the linear formula derived from the back-analysis. The stope span was about five times the pillar width (extraction ratio ~ 83%) and the model height was more than eight times the pillar width. The input data for the ductile models (Figure 5-8) was determined from underground pillar measurements of stress and deformation at the Amandelbult site. The model results are compared to the linear formulae (Equations 5-9 and 5-10) for a range of w/h in Figure 5-33. The strengths shown in the figure were slightly higher than measured at the instrumentation sites, due to the 2D conditions assumed in the model and the equations. The 2D model conditions were accounted for by assuming infinitely long pillars in the equations. The formulae and the model results were also compared to laboratory tests

performed by Spencer and York (1999). These tests were carried out using a cylindrical punch of 25 mm diameter and a foundation cylinder of 80 mm for the diameter and the length. The foundation was confined by a metal ring that was heat-shrunk around the cylinder prior to testing. Both the punch and the foundation were prepared from anorthositic norite pillar and footwall material from the Impala Platinum Mine. The w/h ratio of the punch was varied by changing the height. The boundary condition at the top of the punch was unrealistic and this parameter controls the effective w/h ratio. In addition, the shape of the pillar was different to the normal underground pillars and the ratio in size between the punch and foundation did not adequately represent the underground situation. These issues need to be considered in any comparison between the laboratory results and the strength back-analysis or the modelling. Despite these differences, a remarkable comparison is shown in Figure 5-33. In particular, a good correlation is shown between Equations 5-9 and 5-10 and the numerical results obtained for the ductile material. Both suggest an approximately linear relationship between pillar strength and pillar w/h ratio for the range of pillar sizes in the database.

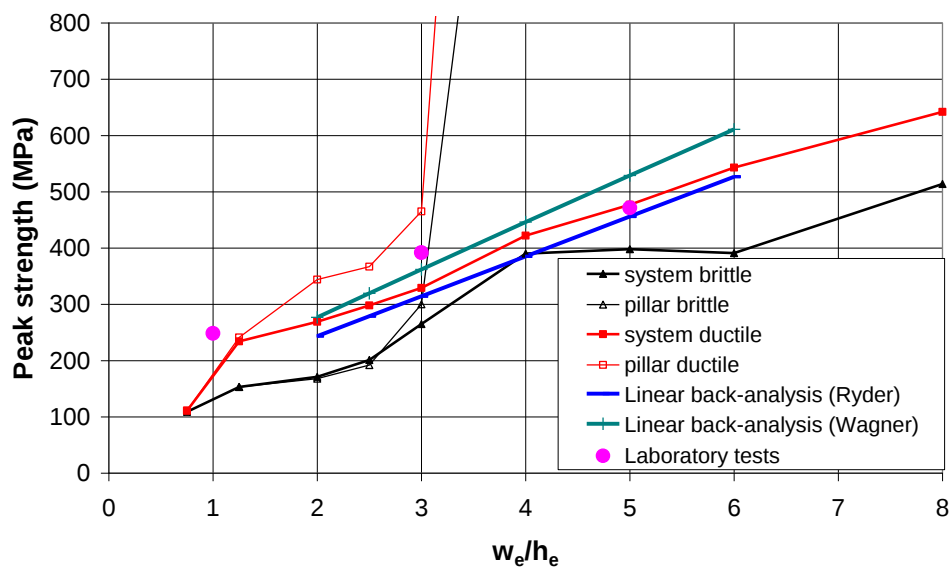


Figure 5-33 Comparison between the strength database, FLAC modelling and laboratory tests performed by Spencer and York (1999)

The modelling results suggest that the pillar systems in the database compare very well with the 'ductile' model. This implies that footwall and/or hangingwall damage would have initiated at a relatively low w/h ratio of about 1.2 – that is, for most of the pillars in the database. The punch tests performed by Spencer and York (1999) suggest that damage occurred in the footwall at a w/h ratio of unity (Figure 5-14). However, a comparison between the laboratory tests and the model results indicates that the foundation damage was less severe in the laboratory sample than predicted by the model for a w/h ratio of 3. It should be noted, however, that the laboratory test was not performed on an infinitely long pillar. The effective w/h ratio also was smaller than indicated in the graph due to the low contact friction angle of the loading platen ($\sim 13^\circ$).

5.5.10 Discussion

The very small standard deviations in the pillar strength analyses are evidence of a good- quality database. The linear formulae provided slightly smaller standard deviations and, therefore, better results for the whole database than the power formula. The numerical models also suggested a linear relationship between pillar strength and w/h ratio up to a w/h ratio of about 8. In addition, the back-analysis performed on the CC3 pillars matched the linear strength parameters calibrated from the whole database very well (Figure 5-29 and Figure 5-30).

The effect of length-to-width ratio was also investigated in the study. As 90% of the pillars in the database have a length of less than 16 m, the range of w/l ratios is limited. Nevertheless, when strengths calculated using the Ryder et al (2005) and Wagner (1974) length corrections are compared, the former correction appears to be slightly conservative but more realistic. This is particularly the case for greater pillar widths and lengths – i.e. the Wagner (1974) length correction appears to overestimate the pillar strength for wide and long pillars. It would appear then that Equation 5-9, with the back-fit parameters provided in Table 5-4, provides the best estimate of pillar strength (at least for the range of w/h_e in the database).

The numerical models suggest that the w/h_e -strength relationship may not be linear at w/h_e ratios greater than about 8 (Figure 5-9). In addition, the

investigation only considered pillars from the Impala Platinum Mine. It is therefore advised that the formula only be used on pillars with similar geotechnical and geomechanical characteristics to those of the pillar systems in the database. The effects of gullies adjacent to pillars and the depth of sidings have not been properly investigated either, and improved w/h ratio estimates may be possible when these factors are better established. A better understanding of the influence of gullies and sidings on pillar behaviour would also allow a more accurate assessment of the effects of pillar length on strength.

5.6 Measured pillar behaviour

The maximum-likelihood statistical back-analysis on a database of failed and unfailed pillars showed that a linear formula best described the effect of pillar w/h ratio on peak pillar strength. This relationship was confirmed by FLAC modelling if failure was allowed to progress into the foundations (hanging- and footwall). The instrumentation programme described in Chapter 4 showed that damage occurred in both the hangingwall and the footwall. The following section describes underground measurements of stress and deformation conducted on individual pillars to establish underground pillar behaviour, including peak and residual strength. The measured peak pillar strengths were compared to the linear strength formula established in Section 5.5.6.

5.6.1 Introduction

A total of six pillars was instrumented at the Amandelbult, Impala and Union sites under a variety of geotechnical conditions. The stress changes were measured using 2D and 3D straincells mounted in the hangingwall above the pillars. Boussinesq evaluations (Poulos and Davis, 1974) show that the ideal height for determining average pillar stress (APS) from a point measurement above a 3 m-wide pillar is 6 m above its centre. At this height the position of the pillar peak stresses during the evolution from intact to failed conditions has little influence on the magnitude of the measurements (Figure 5-34). However, at this height the cell also measures the effects of adjacent pillars and the stope face (Figure 5-35), which are difficult to separate in the final analysis. The installed positions were therefore generally a compromise to minimise errors associated with stress-profile changes during the pillar evolution and adjacent stope face and pillar affects. All of the instruments used to measure pillar behaviour are described in Chapter 4.

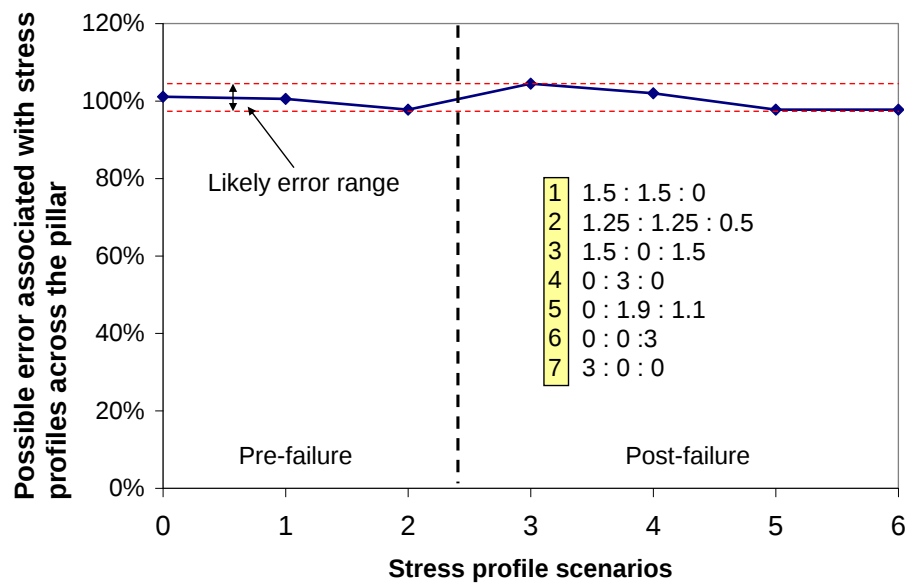


Figure 5-34 Synopsis of potential errors from the point measurement in the hangingwall on the APS, assuming various stress-profile scenarios across a pillar

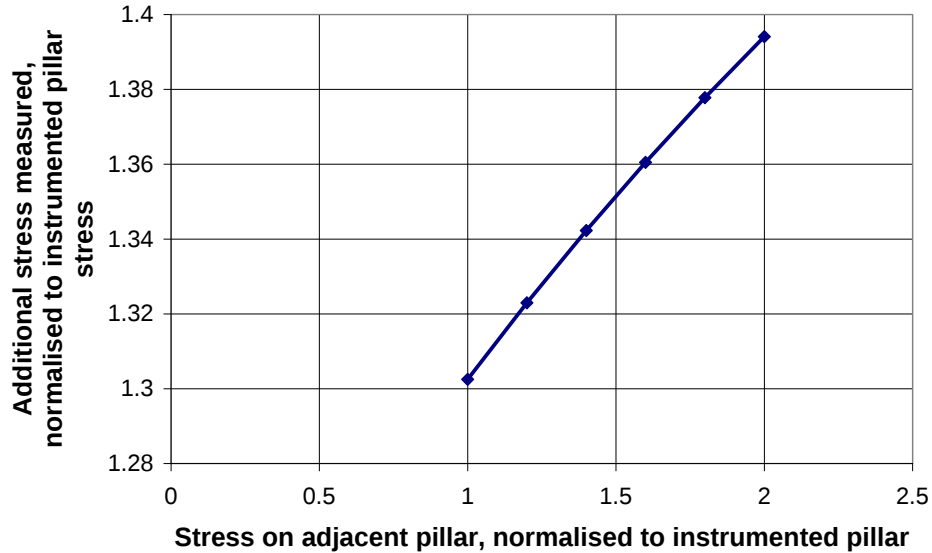


Figure 5-35 Effect of adjacent pillars on the measurement point assuming 4 m-long pillars. One of the adjacent pillars was assumed to be carrying the same stress as the instrumented pillar

5.6.2 Methodology for evaluating stress change measurements

APS was estimated from stress measurements conducted in the hangingwall above the instrumented pillars. Appropriate conversion factors were determined from MinSim models and Boussinesq evaluations (Poulos and Davis, 1974) as described in Appendix G. Comparisons between the measurements and elastic models showed that stress-change measurements had to be evaluated using the 'matrix' modulus of the host rock. However, the field measurements that were affected by microfracturing were evaluated using the methodology shown in Section 3.8. Peak pillar strengths were checked using detailed MinSim models of the stopes and the linear peak pillar strength formula as shown in Section 5.5.6.

5.6.3 Methodology for evaluating residual-strength measurements

Residual-stress profiles were measured at regular intervals from shallow-dipping boreholes drilled across the top of the instrumented pillars. These evaluations were done after the pillars had reached their final residual state, as indicated by the stress change measurements.

Most of the residual stress measurements were made using 2D doorstopper instruments. The results reflect subvertical and horizontal stresses in the plane of the pillar long-axis (Figure 5-36). An inverse matrix of Boussinesq equations (Equation 5-12) was used to extrapolate the pillar stress profile from the subvertical measurements. However, the stress measurements were often conducted between subvertical fractures and joints, which influenced the stress distributions. For this reason the extrapolated stress distributions across the pillars could often not be reasonably replicated mathematically, and a 'best fit' estimation of the measurements was used. The 'best fit' estimation resulted in a smooth pillar profile, which is probably an approximation of the actual profile. An example of a typical matrix grid that was employed in the evaluations is shown in Figure 5-37.

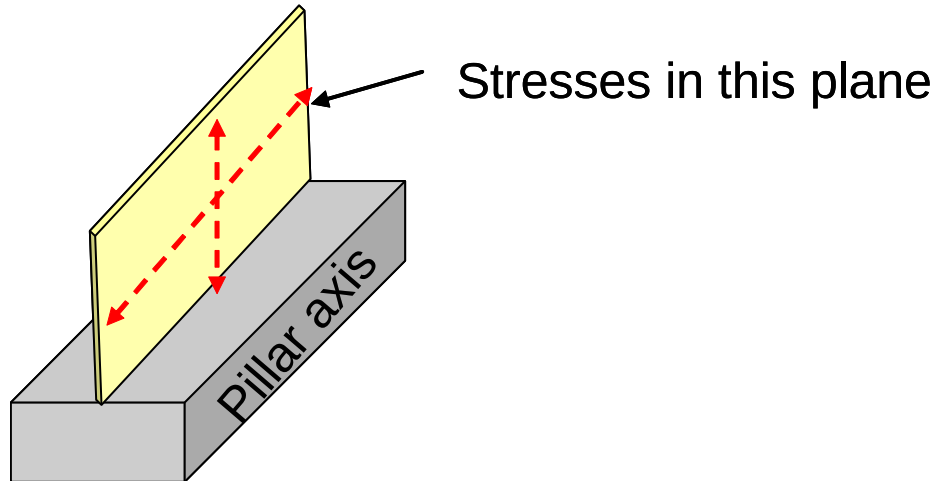


Figure 5-36 Sketch of a pillar showing the plane in which the 2D residual stress measurements were made

$$\sigma_{zz} = \sum_{i=1}^n \left[\frac{3A_i}{2\pi} \times \frac{z_i^3}{(x_i^2 + y_i^2 + z_i^2)^{5/2}} p_{zi} \right] \quad 5-12$$

0.5 m	21 X= -2.32 Y= 2.52	22 X= -1.16 Y= 2.52	23 X= 0 Y= 2.52	24 X= 1.16 Y= 2.52	25 X= 2.32 Y= 2.52	Pillar
	16 X= -2.32 Y= 1.96	17 X= -1.16 Y= 1.96	18 X= 0 Y= 1.96	19 X= 1.16 Y= 1.96	20 X= 2.32 Y= 1.96	
	11 X= -2.32 Y= 1.40	12 X= -1.16 Y= 1.40	13 X= 0 Y= 1.40	14 X= 1.16 Y= 1.40	15 X= 2.32 Y= 1.40	
	6 X= -2.32 Y= 0.84	7 X= -1.16 Y= 0.84	8 X= 0 Y= 0.84	9 X= 1.16 Y= 0.84	10 X= 2.32 Y= 0.84	
	1 X= -2.32 Y= 0.28	2 X= -1.16 Y= 0.28	3 X= 0 Y= 0.28	4 X= 1.16 Y= 0.28	5 X= 2.32 Y= 0.28	
1.0 m						

Figure 5-37 Typical grid configuration used in the Boussinesq evaluations

The average residual strength across the pillar was calculated from the stress profile and, in most cases, was compared to a separate field measurement conducted well above the pillar.

The 'best fit' curve was estimated, assuming the following:

- the APS values estimated from measurements conducted high above the pillars were correct;
- there was a smooth stress profile across the pillars;
- 'joint-channelling' exaggerated stress peaks and troughs at the measurement positions; and
- the 'best fit' profile is approximately an average between the measured stress peaks and troughs.

The measurements were initially compared to the profile provided by a uniform stress distribution across the pillar. This comparison provided an indication of the possible peak stress magnitude at the pillar centre and the stress conditions near the pillar edge. In most cases, the profile produced by a uniform stress was similar to the measurements, which suggested that the stress distribution was almost uniform with a relatively low peak at the pillar centre. The 'best fit' curve was thus a modification of the uniform profile to better represent the measurements. Many of the stress measurements showed an irregular stress profile across the pillar, which could be the effect of 'joint-channelling' or relatively low and high stresses in the pillar itself. The reality was probably a complex interaction of both effects. In some instances, the only reasonable estimation of the 'best fit' curve was to assume that some of the measured stresses had been channelled upwards and these stresses were in fact the pillar stresses. This assumption was only made where a discontinuity was observed in the stress measurement borehole.

The possibility of errors due to instrumentation variation is believed to be small as each cell was inspected and electronically tested prior to installation. A cell was also installed in a 300 mm cube of rock and tested in a press. A maximum error of about 2 MPa (<10%) was established for the cell under these ideal loading conditions.

5.6.4 Shallow-depth good rock mass conditions (Amandelbult site)

5.6.4.1 Amandelbult site: locations of the instruments

The stope sheet shown in Figure 5-38 indicates the approximate locations of the straincells and closure meters used to measure the behaviour of Pillar 1 (P1) and Pillar 2 (P2) at Amandelbult. Relevant face advances are shown by the array of colours in the figure. The site was located at 600 m below surface and the rock mass conditions could be described as very good (Section 2.1). The stress-change cell above P1 was installed about 2.2 m behind the down-dip face in Panel 13-16-2E, while the cells above P2 were installed 1.3 m ahead and 0.82 m behind the face, respectively. All the closure stations were installed ahead of the face.

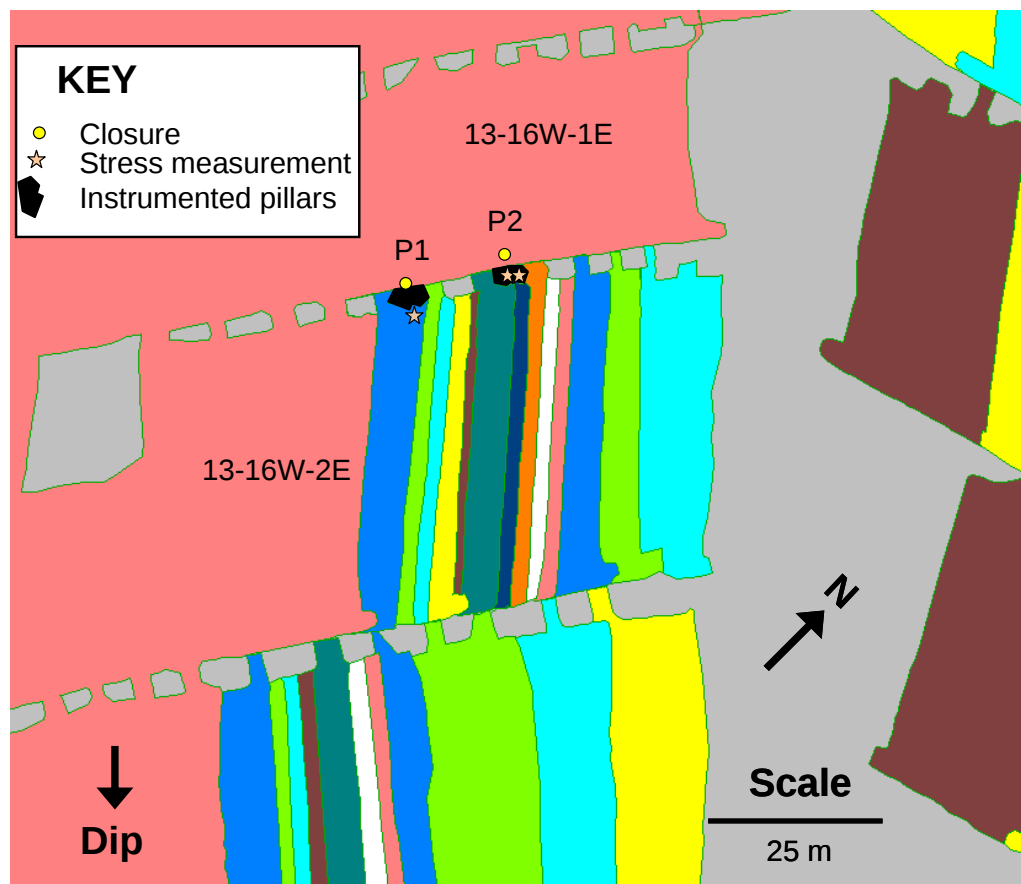


Figure 5-38 Amandelbult site: stope sheet showing the positions of the stress-change cells and closure meters

Section and plan views showing the orientation of the straincell relative to P1 at installation, are shown in Figure 5-39 and Figure 5-40, respectively.

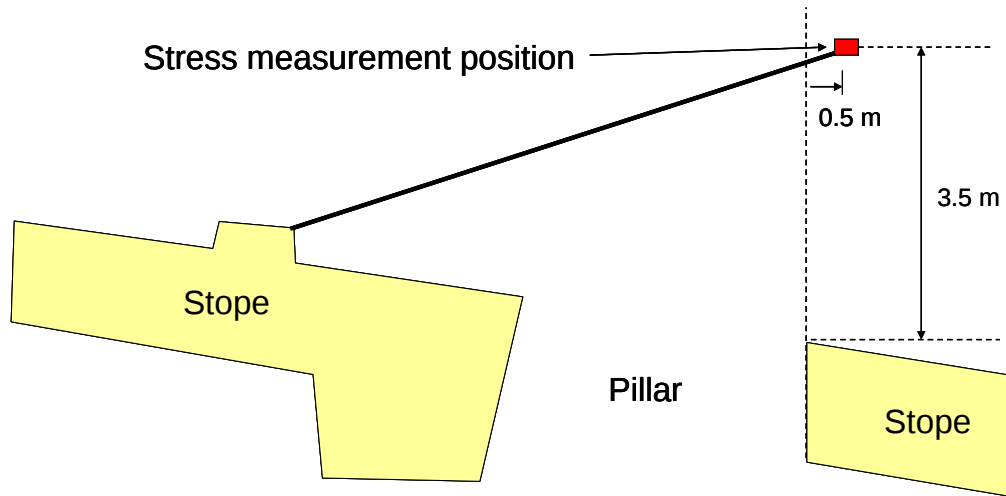


Figure 5-39 Amandelbult P1: section showing the instrumentation position

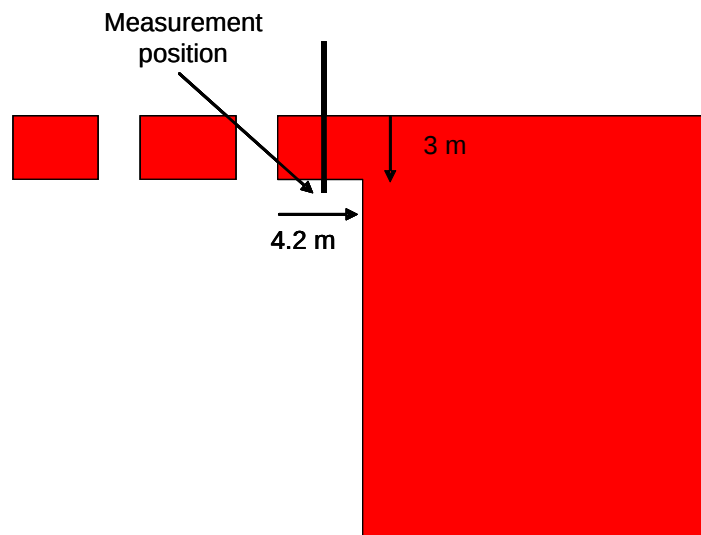


Figure 5-40 Amandelbult P1: plan view of cell position

Two stress-change cells were installed above P2 as follows:

- A 3D CSIRO cell was mounted at a height of 6.5 m above the pillar and 1.3 m ahead of the down-dip face, i.e. prior to pillar formation (1 in Figure 5-41 and Figure 5-42); and
- A 2D doorstopper was mounted at a height of 6.3 m above the pillar and 0.82 m behind the down-dip face (2 in Figure 5-41 and Figure 5-42).

The face positions at the installations of the 3D and 2D cells are shown in Figure 5-42 by the red and dotted lines, respectively.

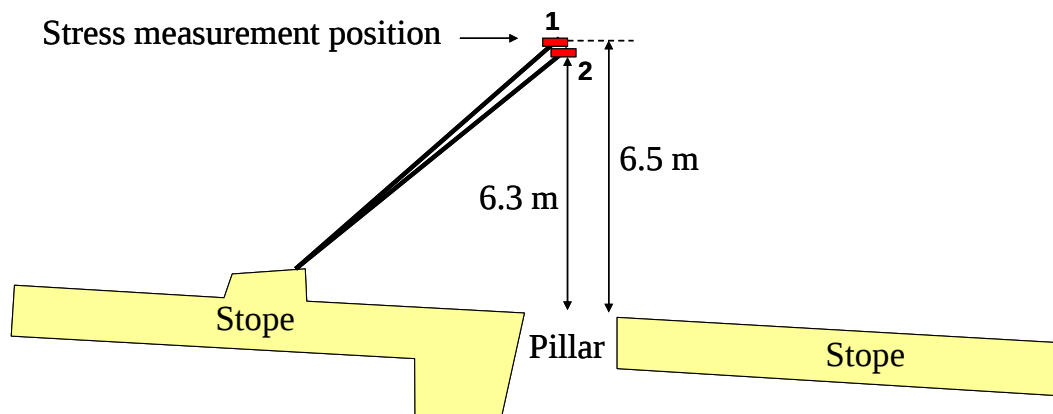


Figure 5-41 Amandelbult P2: section showing the instrumentation positions

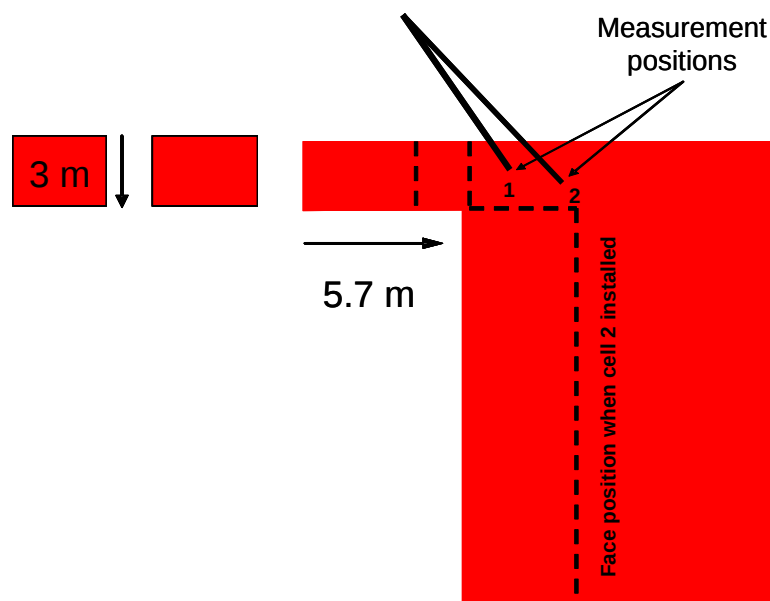


Figure 5-42 Amandelbult P2: plan view of the cell installations

5.6.4.2 Amandelbult site: Pillar description

The nominal size of the pillars at the site was 3 m x 4 m in the dip and strike directions respectively and the stoping width was about 1.1 m. Pillars were cut at the edge of the ASGs and the high sides of the pillars were inclined at between 70° and 80° to prevent fractured slabs from rotating into the ASG (Figure 5-43). On average, the final dimensions of P1 and P2 were 2.5 m x 5.5 m and 3 m x 4.3 m, respectively, both in a stoping width (pillar height) of 1.1 m. A detailed description of the pillar dimensions is provided in Appendix G.

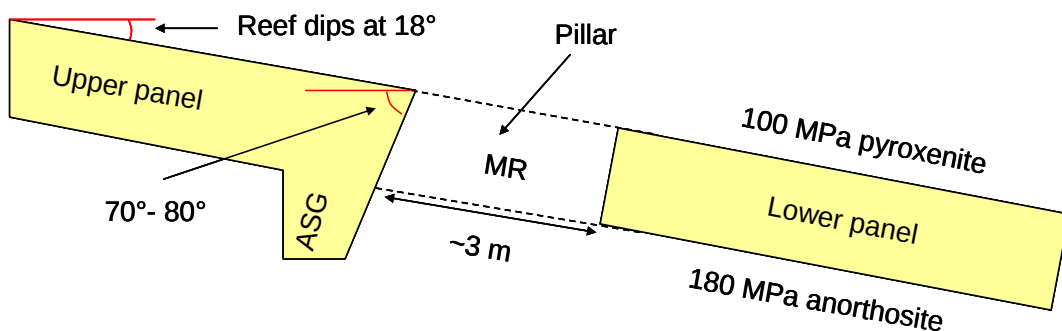


Figure 5-43 Amandelbult site: diagram showing a section through a 'crush' pillar (not to scale)

The immediate footwall of the Merensky Reef at the site is a brittle anorthosite, which has a significantly higher uniaxial compressive strength (~180 MPa) than the pyroxenite reef and hangingwall (~100 MPa). This anorthosite was exposed by the ASGs on the up-dip sides of all the in-panel pillars at the site, i.e. the composite pillars.

5.6.4.3 Amandelbult site: observations

A visible difference in fracturing occurred along the lower abutment of Panel 13-16W-1E up to 16 m ahead of the down-dip panel face when the holing just to the west of P1 was cut to a depth of 2 m (Figure 5-38 and Figure 5-44) – i.e. before the pillar preceding P1 was completely formed. The change in the fracture condition of the abutment ahead of this pillar suggests a stress transfer, probably from the pillar being cut (P0 in Figure 5-44), and indicates that pillar failure is likely to have occurred at this stage. Thus, P0 appears to have failed between

3 m and 6 m behind the face when a 7 m-long stub extended from the face. The stress-change measurements over P1 (the next pillar) showed that this pillar failed at a stub length of 4.2 m, between 0.2 m and 4.2 m behind the face. The measurements over P2 suggested that pillar failure was taking place in line with the face.

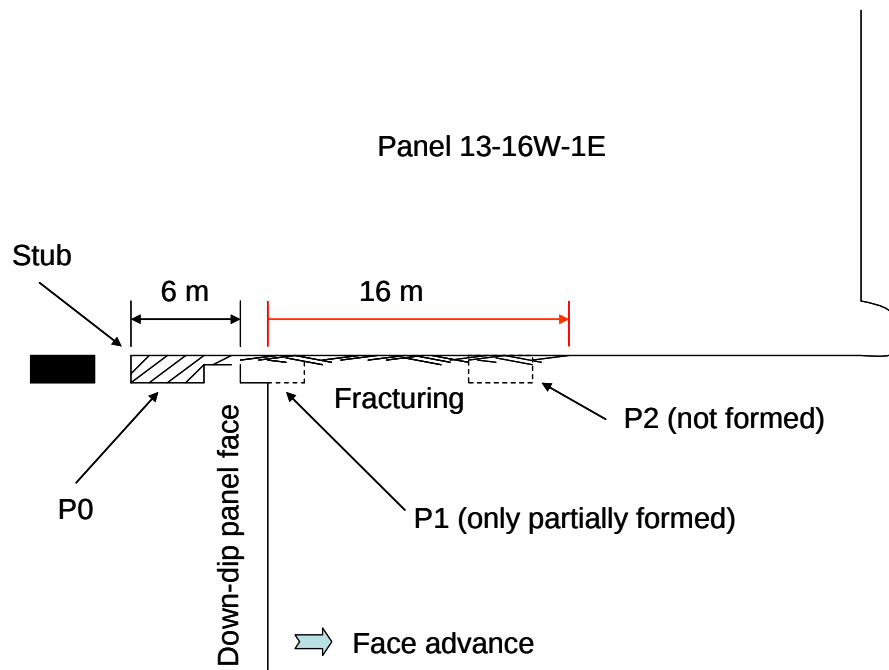


Figure 5-44 Sketch showing the fracturing that occurred when P0 failed

The effects of pillar punching were shown by the rotating bar installed near the base of both pillars (Figure 5-45), as described in Section 4.4. The figure shows that lower rods in both P1 and P2 rotated upwards. Since little or no rotation occurred on the upper rods it appears that the pillars punched into the footwall. The closure measurements set out in Section 4.4 show that while inelastic deformation occurred on the up-dip side of the pillars, almost elastic closure was measured on the down-dip side of these pillars. Thus it appears that there was preferential fracturing on the up-dip side of the pillar. It is possible that the fractures formed below the edge of the abutment before the pillar was formed. At that stage, fractures could only form on the up-dip side. Further inelastic deformations may have preferentially occurred on existing fractures. In addition, the pillars had a higher sidewall on the up-dip side, making the pillar weaker on this side.



Figure 5-45 Amandelbult site: rotation of jumpers cemented into the centre of P2 near the top edge of the pillar and about 1 m vertically below. A = pillar failure, B = three months after pillar failure

5.6.4.4 Amandelbult site: stress-change measurements

In order to relate the stress-change measurements to average pillar stresses, it was assumed that all the surrounding pillars were subject to the same load-deformation characteristics as the pillar being analysed. In this way, maximum and minimum contributions could be quantified and the relationship between measured stress and average pillar stress could be determined using the Boussinesq analytical solution and 3D MinSim numerical models. The evaluation is fully described in Appendix G.

The stress conditions recorded at 3.5 m above P1 (Figure 5-39) as well as the interpreted evolution history of APS are provided as a function of face advance in Figure 5-46. Measurements initiated when the face had advanced 4.2 m – i.e. a 4.2 m long pillar stub had emerged from the down-dip face. At this stage the face was about 2 m from the instrument in plan. The measurements suggest that pillar failure occurred on or before the date of instrumentation installation. However, core-discing occurred while drilling the instrument borehole over the pillar (Figure 5-47) on the same day as the cell was installed, suggesting that there was a high stress over the pillar and that failure had probably not occurred prior to

installation. In addition, the recorded pillar peak stress was comparatively high and it is unlikely that there was a significantly higher stress prior to the instrument installation. “Pillar failure” refers to the strain softening that occurs after peak APS.

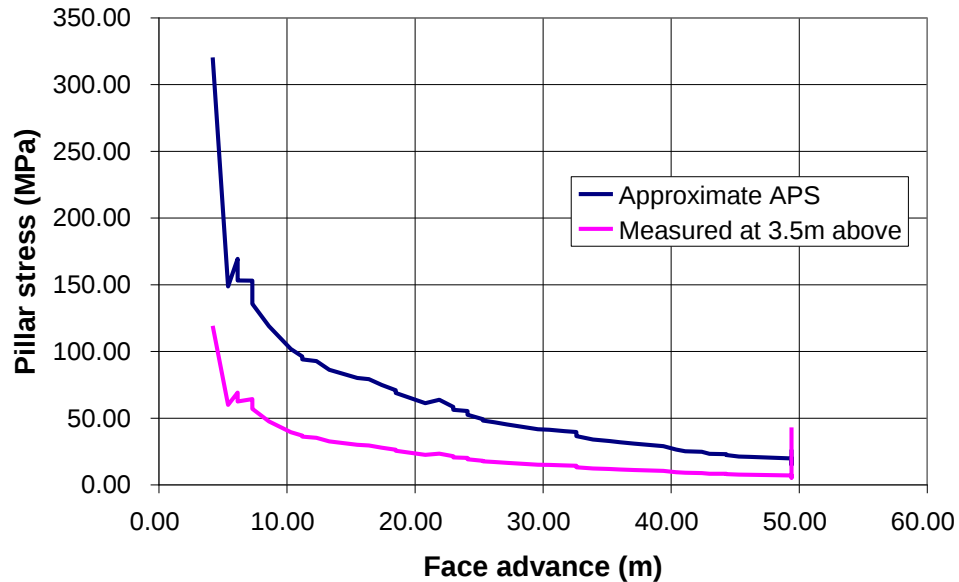


Figure 5-46 Amandelbult P1: stress results as a function of face advance



Figure 5-47 Discing of core from the borehole used to install the straincell above Amandelbult P1

The errors associated with the effects of possible stress profiles on the measurement point were determined using the Boussinesq analytical solution and are described in Appendix G. The range of possible stress errors at the measurement point is shown by the dotted lines in Figure 5-48. The curve indicates that, in real terms, there is high confidence in the residual strength result of about 20 MPa. It should be noted, however, that the measurements

suggested a possible stress drop to 13 MPa after the face reached its stopping position. The peak strength is probably between 240 MPa and 340 MPa, but the MinSim analysis shown in Appendix H suggests that the range is more likely between 240 MPa and 300 MPa.

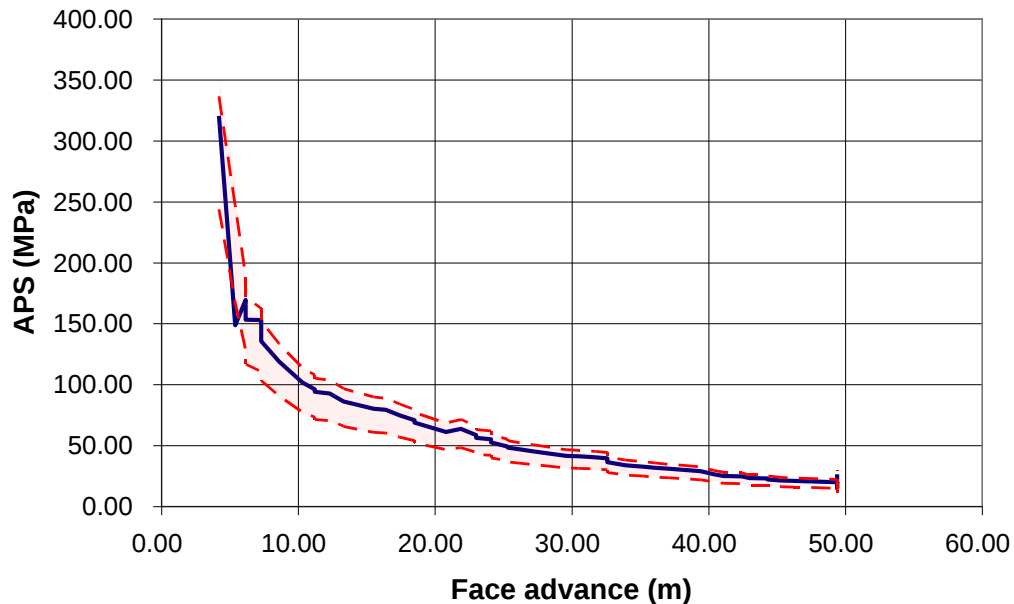


Figure 5-48 Amandelbult P1: stress results showing the possible error due to the range of likely stress profiles across the pillar

Three months after pillar formation the panel was stopped and the APS dropped to 13 MPa (Figure 5-48). Subsequently, however, the measurements showed an increase in APS (Figure 5-49), indicating stress regeneration at a strain of about 82 millistrains or a closure of about 70 mm. The series of field measurements to determine the residual strength and stress profile across the failed pillar (Section 5.6.4.5) was performed when the residual strength was 13 MPa.

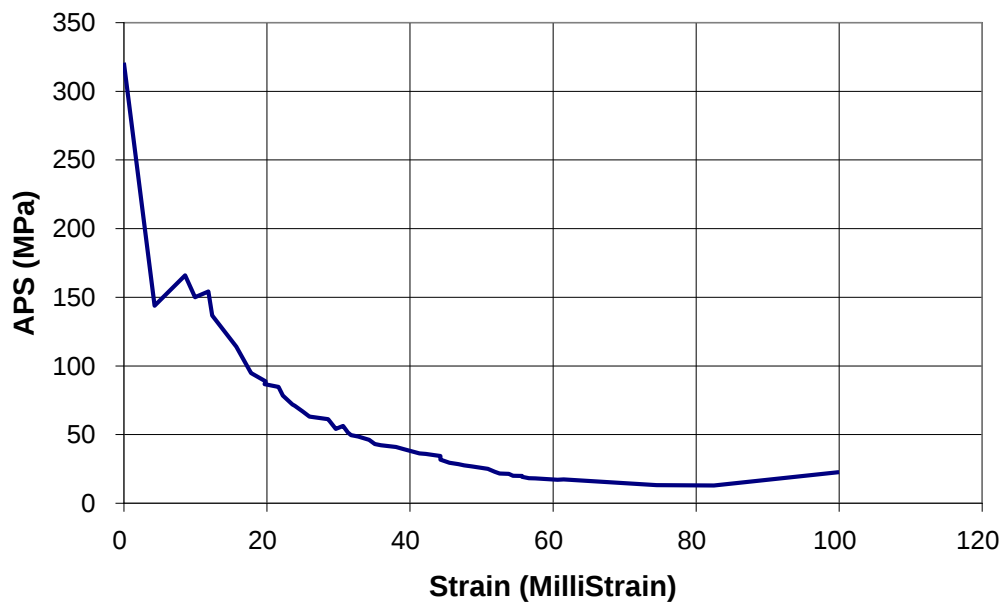


Figure 5-49 Amandelbult P1: stress-strain measurements estimated from the stress measurements conducted at 3.5 m above the pillar and deformation measured adjacent to the pillar

Stress change was measured at two positions over P2 as described above and in Appendix G. The stress effects of the adjacent pillars and the face at the measurements heights (6.5 m and 6.3 m) were significant, and the methodologies applied to extract these influences are also discussed in Appendix G.

The measurements and the estimated pillar behaviour at the two measurement positions were slightly different (Figure 5-50 and Figure 5-51). The best representation of the behaviour of P2 is provided by P2a, as this instrument was located over the centre of the pillar. P2b was installed towards the edge of the pillar and was probably influenced by the adjacent pillar, which had similar dimensions to the monitored pillar. The slightly different behaviours measured by the two instruments show the variability in pillar behaviour. Peak stresses of 276 MPa and 265 MPa were established for P2a and P2b, respectively. These stresses were significantly lower than the peak stress estimated for P1 (~320 MPa) and a MinSim model also suggested a higher peak stress of about 288 MPa if failure occurred when the pillar was a 5 m-long stub. Figure 5-50

shows that failure occurred just behind the face, indicating that the measured stresses should be considered an upper bound to the pillar strength.

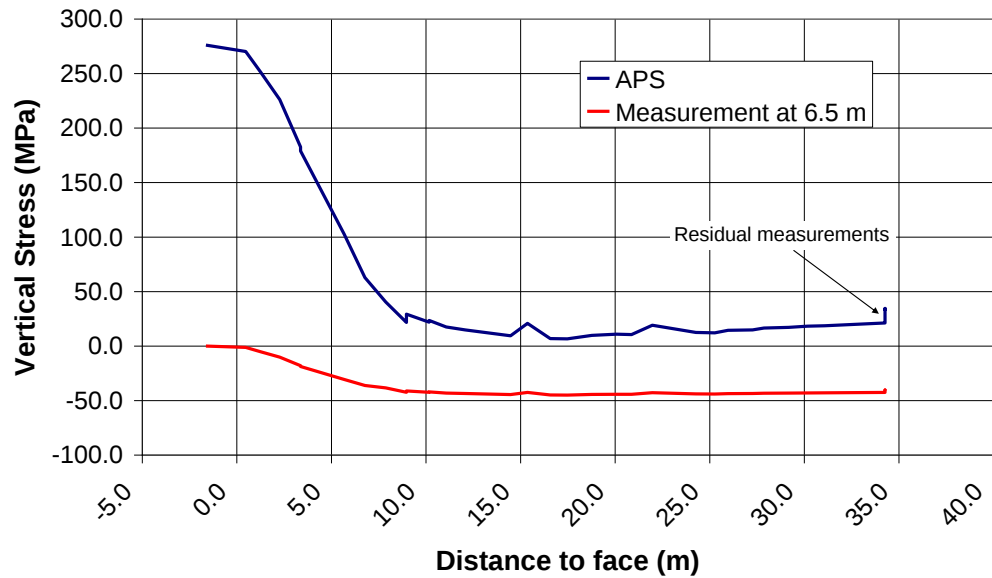


Figure 5-50 Amandelbult P2a stress results plotted against face advance

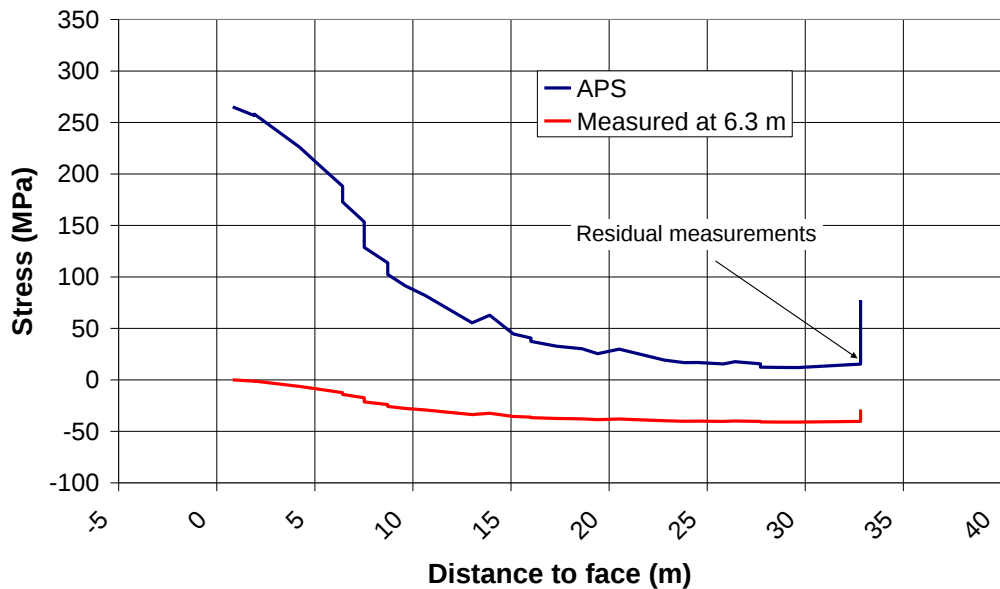


Figure 5-51 Amandelbult P2b stress results plotted against face advance

Residual strengths of between 7 MPa and 22 MPa were estimated from the measurement position of P2a (Figure 5-50). However, P2b (Figure 5-51) indicated a minimum residual strength of 12 MPa, which was more consistent with the minimum measured over P1. The stress drop with face advance in P2b was also less steep than in P2a, probably due to its position relative to the adjacent pillar. As no field measurements of vertical stress were conducted at the time of instrument installation, the peak strength was extrapolated from the residual strength measurements performed at the positions shown in the graphs as “residual measurements” (Figure 5-50 and Figure 5-51). The residual measurements are described in more detail in Section 5.6.4.5.

During the installation of the straincells, fairly high stress conditions were indicated by core-discing (Figure 5-52) that occurred behind the 3D cell installation at about 5.7 m above P2. The core from the original small hole was not fractured, but the stress concentrations of the small hole on the larger over-core were sufficient to cause the observed discing. This discing effectively prevented 3D field stress measurements from being done.



Figure 5-52 Core-discing that occurred about 5.5 m above the proposed P2a position, 2 m ahead of the face

The stress curve shown in Figure 5-50 is considered to most accurately describe the actual pillar behaviour due to the position of the straincell. In addition, 3D straincell results do not have to be adjusted for the stress acting along the borehole axis as is required for the 2D measurements. Small errors may have occurred in some of the 2D measurements due to incorrect estimations of these stresses. The equations applied in the stress calculations are described in Appendix B.

Pillar strain was estimated from closure measurements conducted in the ASG about 1 m from the up-dip edge of the pillar (Figure 5-38). Unfortunately, these measurements were shown to include inelastic deformation, probably due to fracturing in the footwall (Gauge 3 in Section 4.4.1). The stress-strain curve for the post-failure behaviour of the pillar has been estimated from these closure measurements and the extrapolated stress-change measurements at P2a in Figure 5-53. The same closure measurements were used to determine a stress-strain curve for the P2b results in Figure 5-54.

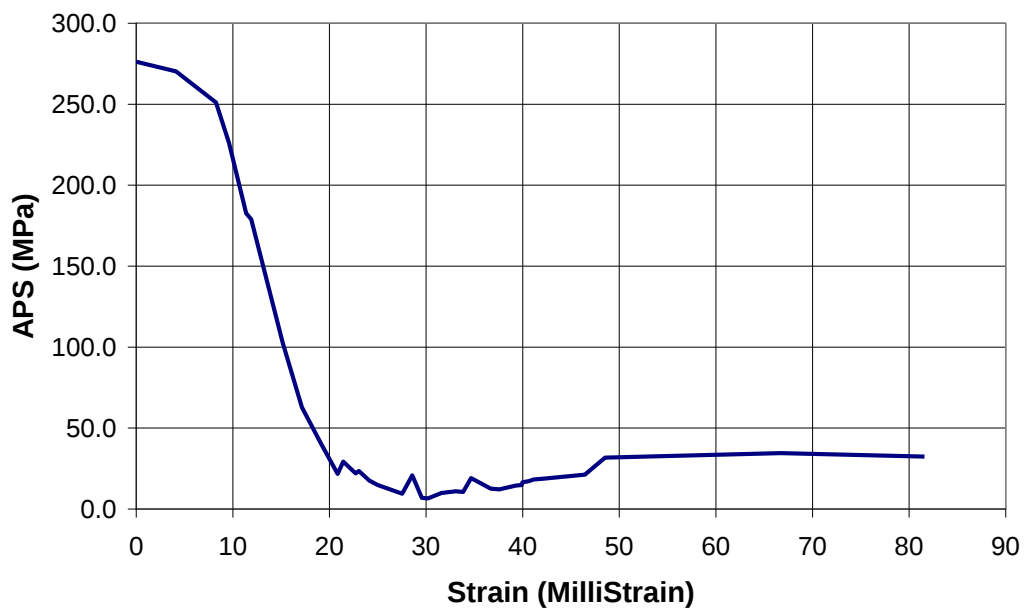


Figure 5-53 Amandelbult P2a: stress-strain measurements estimated from 3D stress measurements conducted at 6.5 m above the pillar and deformation measured adjacent to the pillar

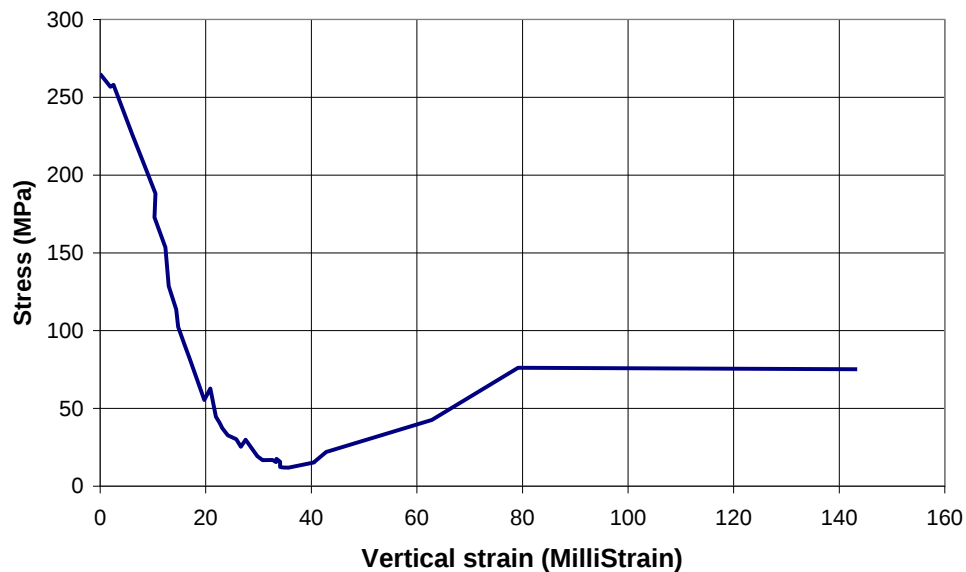


Figure 5-54 Amandelbult P2b: stress-strain measurements estimated from 2D stress measurements conducted at 6.3 m above the pillar and deformation measured adjacent to the pillar

The linear peak strength formula (Section 5.5.6) suggests a strength of 213 MPa for a 3 m-wide pillar without a siding. This strength is significantly less than was shown by the measurements at the Amandelbult site. However, a more comparable strength of 245 MPa is predicted if the gully is ignored in the formula. This strength seems to suggest that little foundation damage occurred at the site, so that the footwall anorthosite acted almost as a solid loading platen, thus minimising the effects of the gully. Previous back-analysis of peak pillar strength (Watson et al, 2007b) also showed higher pillar strengths for Amandelbult (246 MPa to 347 MPa for 3 m- to 4 m-wide pillars without sidings). The maximum likelihood back-analysis of peak pillar strength in Section 5.3 was conducted solely on pillars located at Impala Platinum Mine and thus the underestimation of the pillar strength at the Amandelbult site could be the result of regional differences in pillar strength. This variability in strength may be related to geological conditions but this has not been studied in detail.

At all three measurement positions, the lowest residual strength was measured when there was a reduction in closure rate, as shown in Figure 5-55 for P2b. However, the reduction in closure does not appear to be the only influence on the residual strength as strain hardening initiated before the closure increased.

However, when the stope from the adjacent raise line started influencing the closure rate in Panel 2E, a significant increase in residual strength was observed. As a result of the observed relationship between closure rate and stress increase, it is concluded that the closure rate may have some influence on the residual strength.

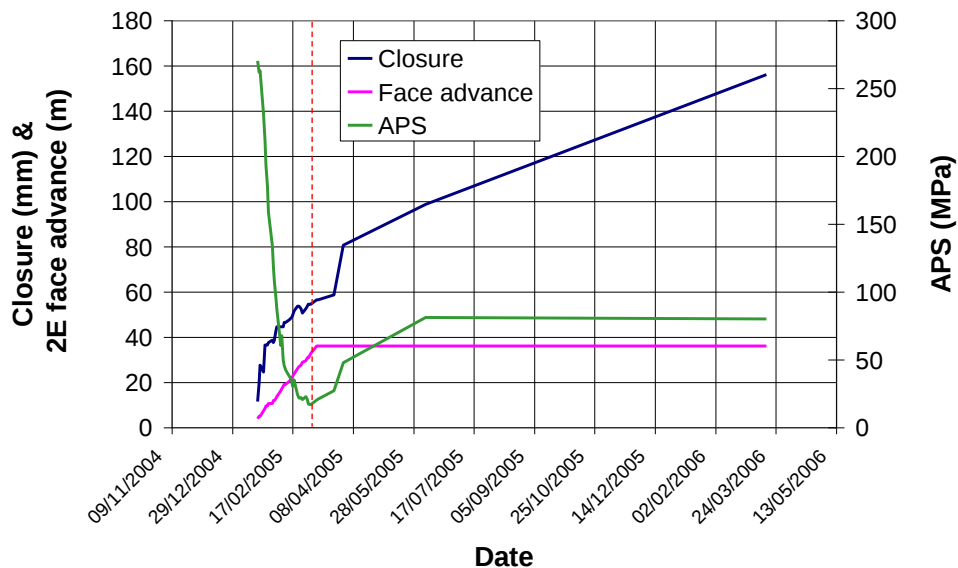


Figure 5-55 Amandelbult P2b: comparison between closure, face advance and APS

The increase in residual strength (stress regeneration) on the two pillars occurred at different dates and strains, suggesting that the convergence resulting from the approach of the adjacent stope was not the only influence in the strain hardening. A greater correlation was observed for stope- and gully-cleaning operations. The initial strain hardening in P2a slowed down when the rock in the ASG was removed (~50 m£ in Figure 5-53). The cleaning of the gully and also the down-dip panel would have reduced confinement on the pillar. A pillar burst occurred previously at Amandelbult during gully-cleaning operations (Watson et al, 2007). At the time, the gully was being scraped/emptied of the ore that confined the pillar for several days prior to the cleaning operation.

The profile generated by the 2D cell was less sensitive to the cleaning operations but also showed stress regeneration and a reversal of this trend when the loose rock on the down-dip side of the pillar was removed (Figure 5-54). The reason for

the large difference in the final strength prediction between P2a and P2b is not clear. There is, however, some uncertainty about the 2D measurements because of the location of the instrument and possible errors in the borehole-axis stress estimations. As a good correlation exists between P2a and P2b up to about 60 m ϵ , the initial part of the P2b curve is considered reasonable. The latter part of P2a probably correctly reflects the final residual pillar strength conditions.

The measurements show a higher peak strength for P1 than P2. A reason for the strength difference could be explained by the comparatively greater brittleness of the anorthosite exposed in the down-dip side of P2. Since P1 was cut slightly in the hangingwall, anorthosite was only exposed on the up-dip side of this pillar. For this reason P2 could be considered more brittle than P1. The modelling in Section 5.4 suggests that a more ductile pillar is stronger. The influence of the behaviour of one rock type on the other may also have caused premature failure in P2 and this could be another explanation for the difference in pillar strengths between P1 and P2.

Pillar stress regeneration was measured by all three of the straincells. However, the extent of the increases and the strains at which these increases initiated were different (Figure 5-56). P2 had a generally steeper strain-softening profile and an earlier stress-regeneration initiation than P1. The average residual strength of the 'crush' pillars during the working life of the panel was about 60 MPa, with P1 being slightly higher than P2 due to the comparatively more ductile strain-softening profile. The generally more gentle post-failure stress drop shown by P1 is further evidence of the effect of anorthosite on pillar brittleness and indicates that the percentage anorthosite within a pillar may influence its post-failure behaviour.

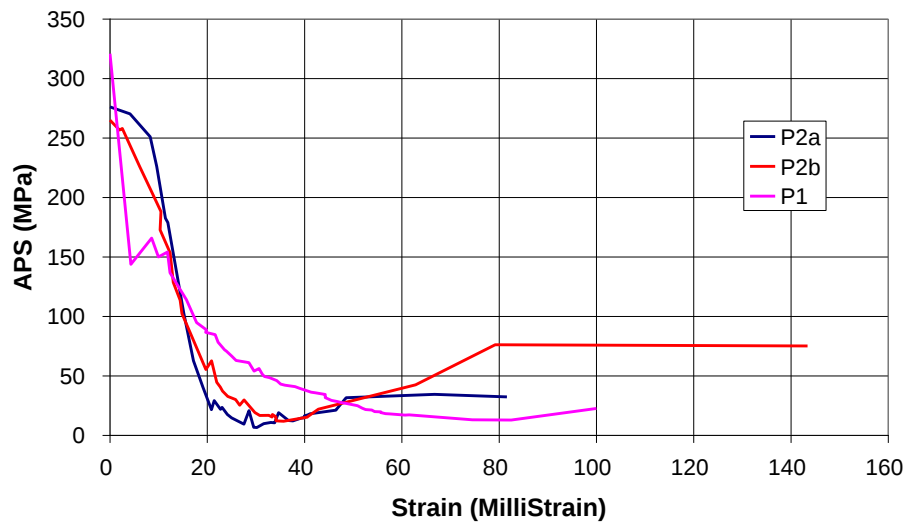


Figure 5-56 Stress-strain curves derived from the three measurement positions over the two Amandelbult pillars (P1 and P2)

Creep tests were conducted on the strain-gauge glues over a six-month period to see if the measured stress regeneration was an artefact of glue behaviour over an extended monitoring period. However, the results did not show any glue creep. In addition, the cells were installed with two different glues and both indicated stress regeneration. It is therefore concluded that stress regeneration actually occurred. It is possible, though that the magnitude of the stress regeneration was exaggerated in the 2D cells (P1 and P2b) because of the effects of horizontal stress (Appendix B). Additional horizontal stress was probably generated in a direction parallel to the borehole axis from the generation of fractures above the pillars (Section 4.3). This effect would have been strongest for P1 because the direction of the hole axis was perpendicular to the pillar line. P2b was at an acute angle to this line. Since P2a was a 3D cell, the fracture-generated stresses did not influence the vertical stress measurements.

A FLAC model using the large strain option and the parameters shown in Table 5-7 was run to simulate the stress regeneration in P1. The input parameters were calibrated from the measured stress-strain curve. It was found that stress regeneration could only be simulated when there was minimal damage in the foundations. The model results are compared to the P1 measurements in Figure 5-57. In reality, the increase in residual stress is probably due to sliding of relatively large blocks and wedges at the centre of the pillar.

Table 5-7 Material and FLAC model properties (calibrated from Amandelbult P1)

Cohesion		Friction		Dilation	
ε_p (m ε)	C_o (MPa)	ε_p (m ε)	ϕ_{res}	ε_p (m ε)	ψ_{res}
0	35	0	24	0	0
2.5	33	2.5	41	2.5	26
5.5	26	5.5	42	15.0	5
8.0	13	8.0	43	1000	5
15.0	5	15.0	42		
30.0	0	1000	42		
1000	0				

C_o = cohesion, ϕ_{res} = residual internal friction angle, ε_p = plastic shear strain, ψ_{res} = residual dilation angle.

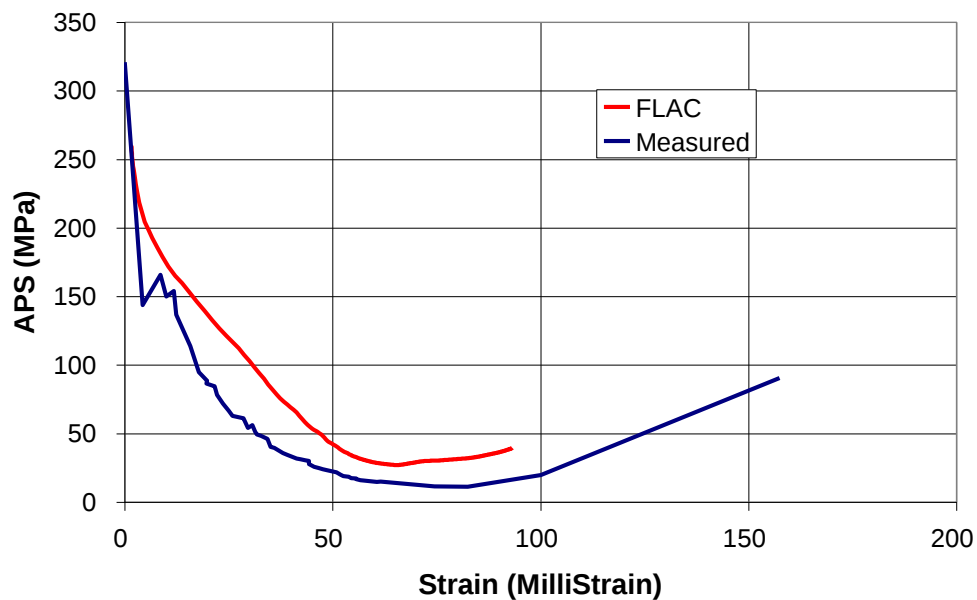


Figure 5-57 Comparison between the measured and FLAC generated Stress-strain curves for Amandelbult P1. The model was conducted using large strain and the parameters shown in Table 5-7

5.6.4.5 Amandelbult site: residual stress measurements

The measurements described in this section were conducted to establish stress profiles across crush pillars that had reached residual strength. The results were also used to confirm the residual strengths determined from the stress change measurements.

2D residual stress measurements were made at regular intervals across the top of the two instrumented pillars (Amandelbult P1 and P2), after the face of the panel below had reached its limit. The measurements were conducted across the narrow section of the pillars from a borehole drilled at about the centre of the long axis of the pillars. These boreholes were drilled up at 23° to the strata (5° above horizontal) as shown in Figure 5-58.

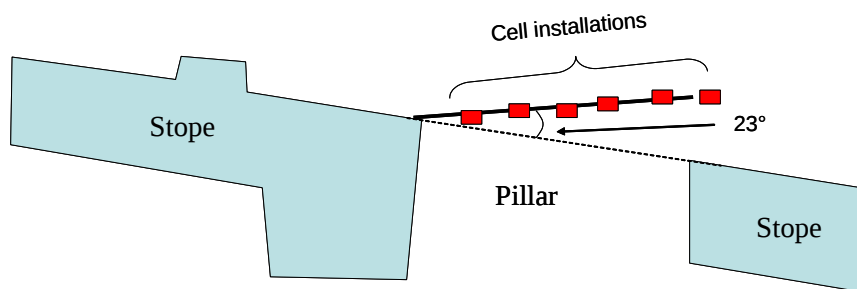


Figure 5-58 Section showing the residual stress profile measurement positions over Pillars 1 and 2

The results of the stress profile measurements over P1 are shown in Table 5-8. Measurements were conducted over the 2.5 m-wide pillar and over the stope on the down-dip side of the pillar. The errors in the strain readings (Table 5-8) are small and the results are therefore considered reliable (Appendix B). The method of calculating the quality of the stress measurements (error) is discussed in Appendix B. One of the gauges on each of two cells was damaged and thus error calculations could not be made on these cells.

Table 5-8 Amandelbult P1: stress profile measurements. Error estimates could not be made in cells with only three operational gauges

Distance from up-dip pillar edge (m)	Height above pillar (m)	Error in strain measure. (%)	Vertical stress (MPa)
0.6	0.23	-	9.3
1.1	0.43	0.1	45.6
1.8	0.70	-	12.1
2.6	1.02	2.1	8.6
3.1	1.21	6.2	11.6

The measured profile (Figure 5-59) could not be replicated using a reasonable inverse matrix of Boussinesq equations. Thus a 'best fit' solution was applied (as described in Section 5.6.3). The results of this evaluation and the resultant pillar profile are also shown in Figure 5-59. The analysis suggests a peak stress of 45 MPa at the centre of the pillar and an average residual strength of about 13 MPa. The effect of a uniform distribution of 13 MPa across the pillar on the field measurement positions is also shown in Figure 5-59 for comparison. The fairly good correlation of this back-analysed profile on the measurements confirms a fairly flat distribution of stress in the pillar with an off-centre peak towards the up-dip side. The profile suggests exponential strengthening at the pillar edges and a relatively low peak at the centre.

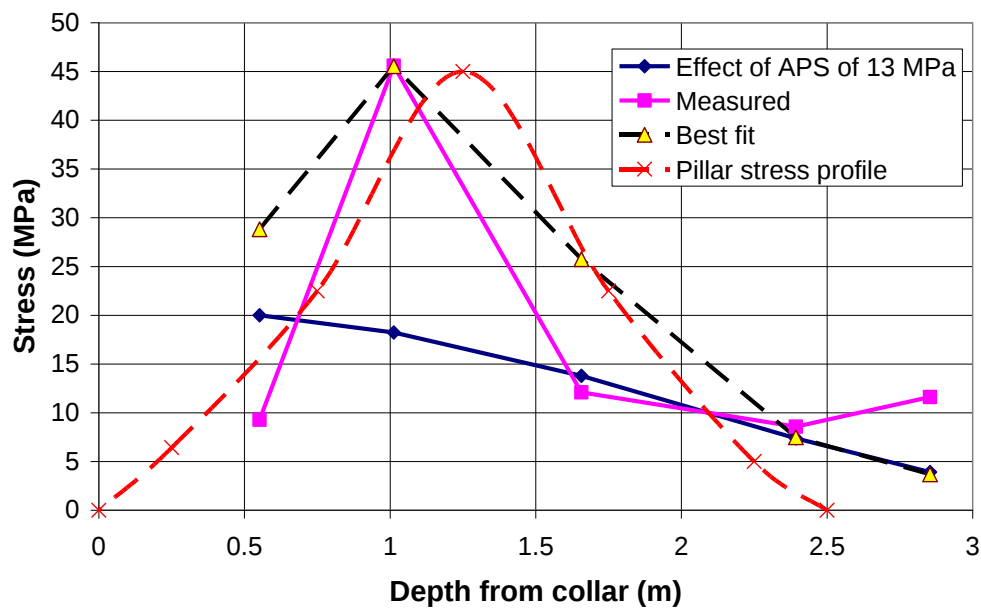


Figure 5-59 Amandelbult P1: showing the measured, ‘best fit’, uniform (APS = 13 MPa) and estimated pillar stress profiles

The average residual strength was similar to the pillar stress calculated from a field measurement conducted at about 4 m above the pillar. The ‘best-fit’ profile was used to back calculate the stress to this field measurement position and the result is compared to the field measurement in Table 5-9. The calculated and measured stresses are in good agreement, confirming the validity of the profile analysis.

Table 5-9 Amandelbult P1: comparison between the field-measured and back-analysed stresses (from ‘Best fit’ curve in Figure 5-59)

Height above pillar (m)	Measured stress (MPa)	Error in strain measurement (%)	Back analysed stress (MPa)
4.0	5.37	11.6	5.96

Stress channelling between vertical ‘stress-fractures’ and joints (‘joint-channelling’) caused additional complications. An Elfen discrete element model was run to determine the effects of 1.2 m-long vertical joints or fractures above a pillar on the stress measurements conducted 2.5 m above the pillar (Figure 5-60).

The model showed that the stress was channelled between the discontinuities and the stress condition at the measurement positions was thus higher and similar to a stress condition closer to the pillar in the absence of vertical discontinuities. These variations would have been easier to evaluate if a greater number of measurements had been made.

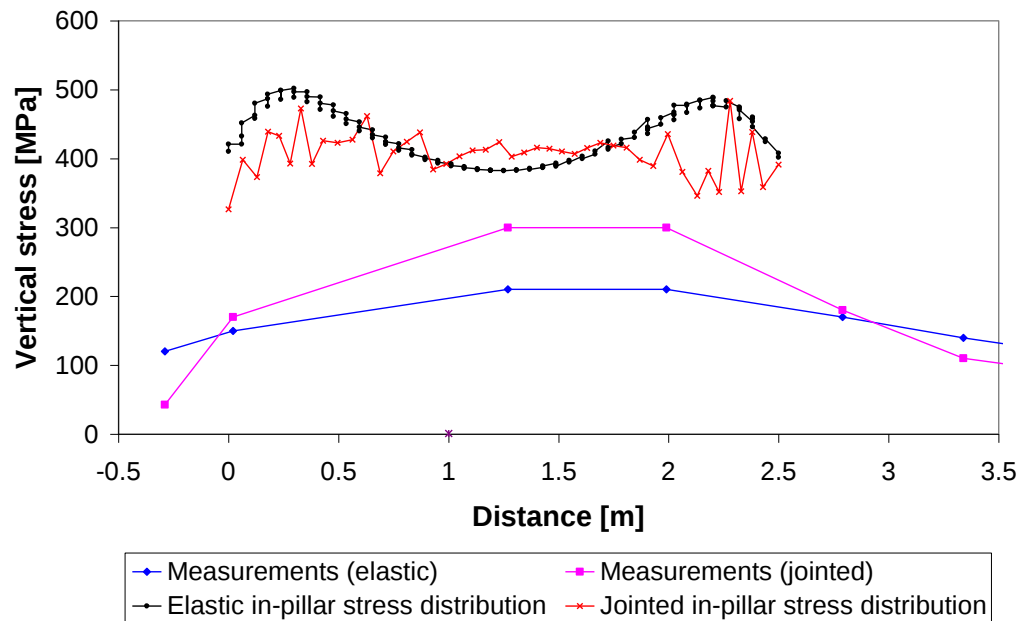


Figure 5-60 Elfen model showing the effects of 1.2 m-long vertical fractures above a pillar (jointed) on the measured stress about 2.5 m above a pillar

The 'best-fit' curve took cognisance of both the evidence of a relatively uniform stress condition as well as the peak measured near the centre. The high and low stress measurements were considered to be exaggerated as a result of channelling effects. Thus the profile that is likely to have occurred in the absence of stress channelling is represented by the 'best-fit' curve. A full description of the evaluation process used to determine the stress profile from the measurements is provided in Appendix G. The results of the 'best fit' stress curve are shown in 3D in Figure 5-61. The effects of the ASG are visible to the left of the diagram.

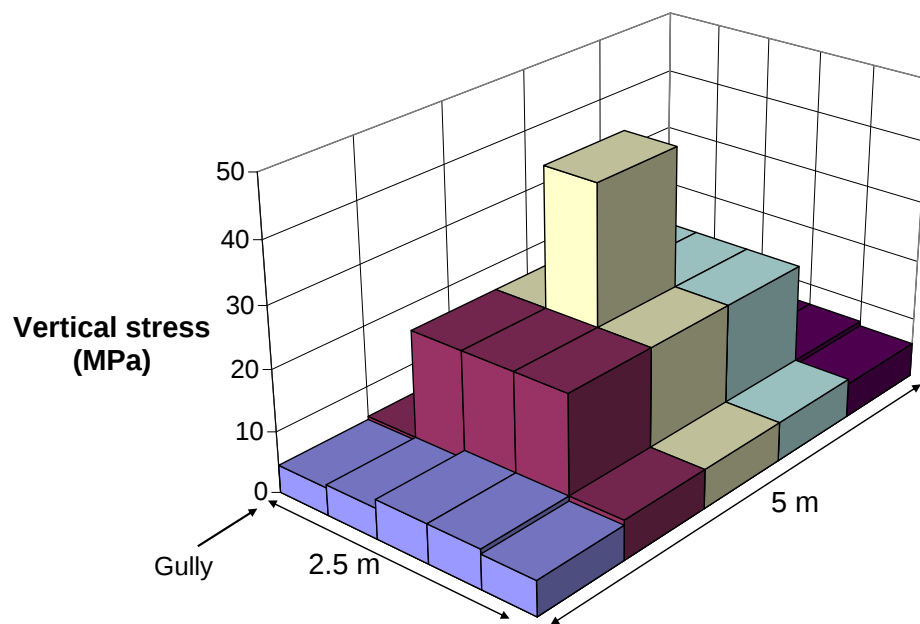


Figure 5-61 Amandelbult P1: 3D stress distribution (from ‘best-fit’ curve in Figure 5-59)

The results of the stress profile measurements conducted over P2 are shown in Table 5-10. P2 was evaluated in a similar way to P1. The results of a back analysed uniform profile of 22 MPa across the pillar are compared to the measurements in Figure 5-62. The first measurement was conducted too close to the pillar for proper Boussinesq evaluation. Also, this measurement had an unacceptably high error (higher than 10%) and was therefore discarded for the purposes of the pillar stress profile evaluation. The second and third measurements were about 19 MPa and 8 MPa higher than the back-analysed uniform profile, respectively. The difference between the measurements and the evaluation suggests that the stress profile across the pillar is not flat and that it probably has a significant peak at the centre of the pillar. The second and third measurements could only be replicated with a reasonable curve if an APS of 22 MPa was assumed for the ‘best-fit’ curve.

Table 5-10 Amandelbult P2: stress profile measurements. The triaxial cell result at 1.5 m is unreliable

Distance down-dip of pillar edge (m)	Height above pillar (m)	Error in strain measure. (%)	Vertical stress (MPa)
1.4	0.59	14.1	18.1
1.8	0.78	4.9	42.4
2.3	0.98	5.6	27.1

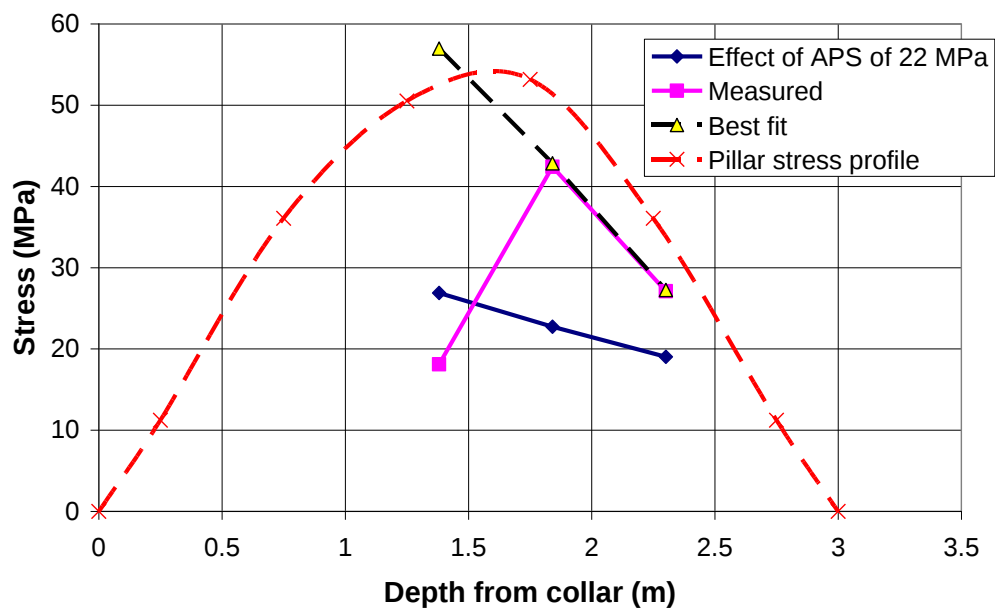


Figure 5-62 Amandelbult P2: comparison between the uniform (APS = 33 MPa), measured, 'best fit' and the estimated pillar stress profiles

Again the back-analysed pillar stress profile shown in Figure 5-62 is only a rough approximation as it was based on only two measurements. Figure 5-62 and Figure 5-63 show a peak stress of about 53 MPa, which is higher than P1.

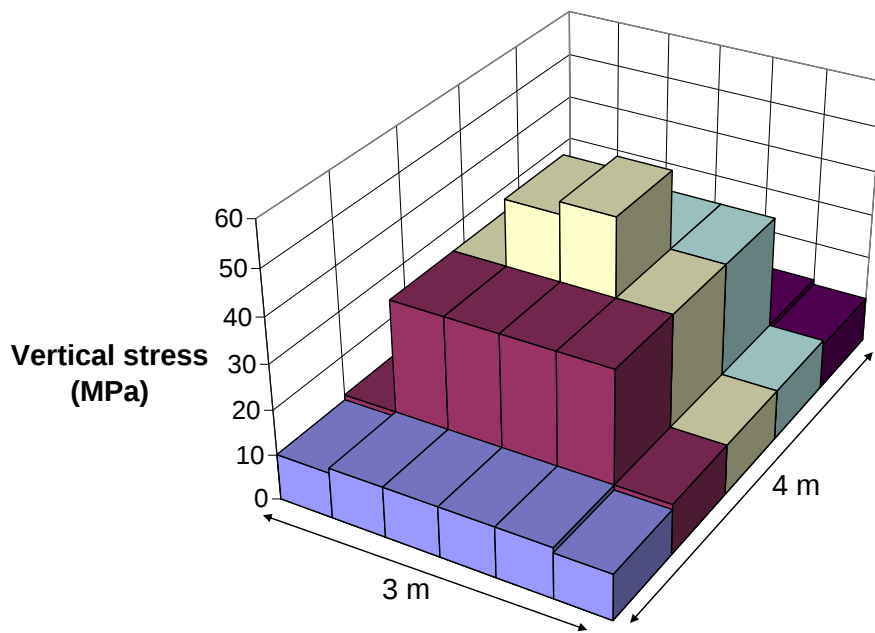


Figure 5-63 Amandelbult P2: 3D stress distribution (from ‘best-fit’ curve in Figure 5-62)

The residual stress profile measurements over both pillars show a relatively low stress at the centre of the pillar at the time of the measurements. However, P2 had a relatively higher residual strength and also a higher peak than P1. The profile measurements were conducted prior to the cleaning of the gully or the down-dip panel adjacent to P2. The higher average residual and peak stress at the centre of the pillar was probably the influence of the confinement of the rock fragments around the pillar. The stress change measurements show that when these fragments were removed there was a drop in the residual strength, but not as low as P1 since P2 was slightly wider. The higher anorthosite content in P2 might also have influenced the pillar behaviour and final strength.

As the measurements were affected by joints, a much larger number of measurements should have been conducted to produce a reasonable stress profile. The limited number of measurements over both pillars and especially over P2 suggests that the profiles should be considered an approximation.

5.6.5 Intermediate depth good rock mass conditions (Impala site)

5.6.5.1 Impala site description

The Impala site was located at a depth of 1100 m below surface and the rock mass conditions could be described as very good (Section 2.2). Measurements were conducted on four different size pillars, marked P1 to P3 and S1 in Figure 5-64. Panel 7s was advanced to the position indicated by the red in Figure 5-64 before mining was initiated in Panel 8s. This advanced position provided an opportunity to install instrumentation over the designated pillars prior to pillar formation. The array of colours in the figure show the monthly face advances. The panels in red, up-dip and to the north of the instrumentation site were mined to completion prior to any mining in Panels 7s and 8s. The footwall entrance into the stope is in Panel 8s. Closure was measured on the up- and down-dip sides of Pillars P2 and P3 and on the down-dip side of P1. Closure was not measured adjacent to S1.

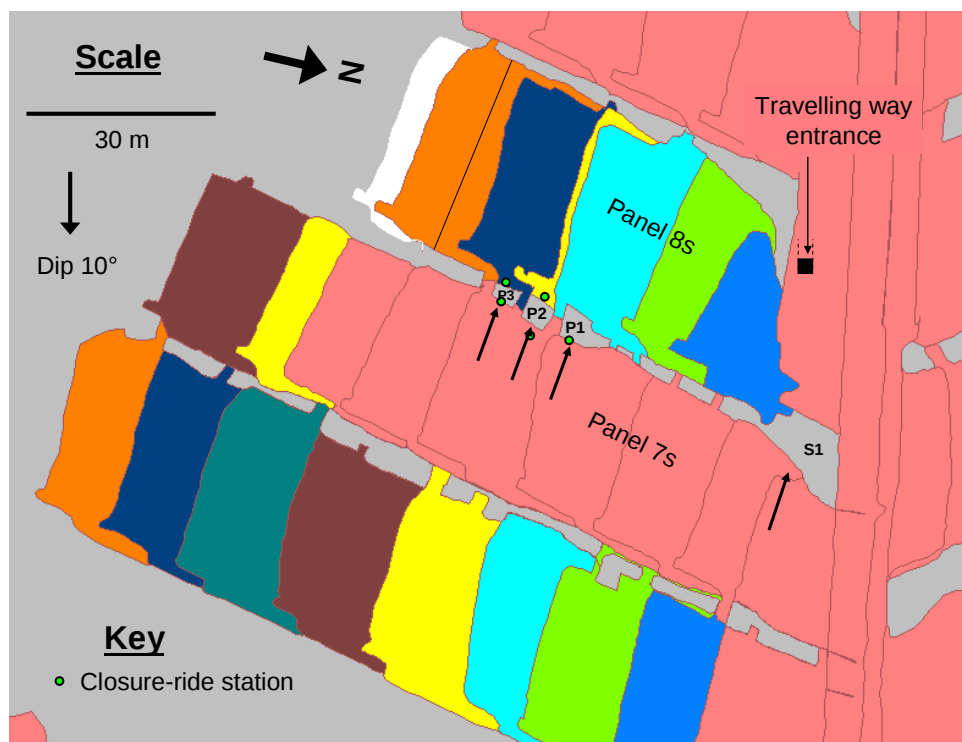


Figure 5-64 Impala site: stope sheet showing the monitored pillars and the travelling way entrance

5.6.5.2 Impala site: locations of the instrumentation

Stress change measurements were conducted at 5.28 m above and 0.74 m from P1, as shown in Figure 5-65. A field stress measurement was conducted in a separate borehole and the stress adjusted to provide a starting stress.

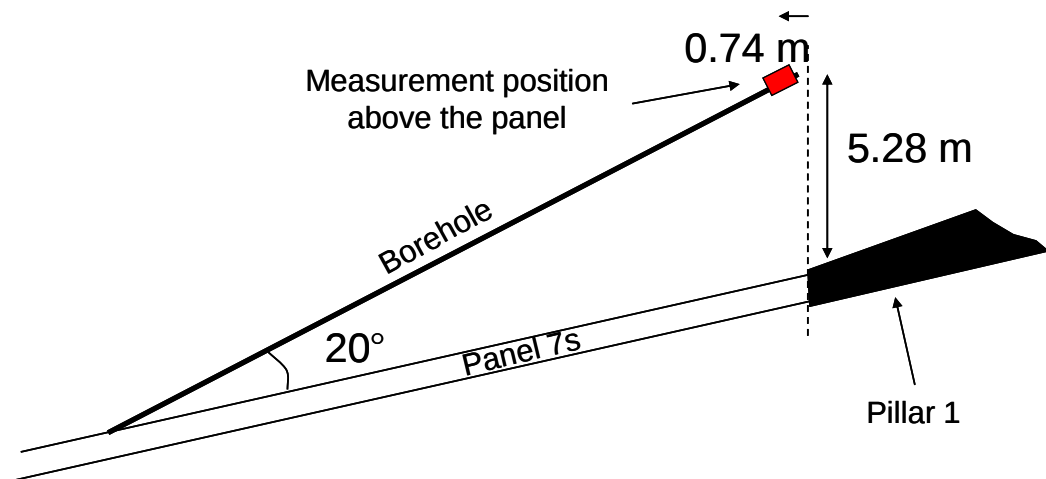


Figure 5-65 Impala site: section showing the straincell position above P1 (not drawn to scale)

Measurements over P2 were conducted from a hole drilled through the centre of the pillar from the haulage below (Figure 5-66) and from a shallow-dipping borehole drilled from the stope (Figure 5-67). No field measurements were possible in the steeply dipping borehole because of the relatively high horizontal stress, but several 2D measurements were made in the shallow-dipping borehole and in an adjacent, parallel hole shown in Figure 5-67. Unfortunately, the cable extending from the CSIRO cell (Figure 5-66) to the haulage below the stope was damaged during pillar failure and no further readings were possible. However, the doorstopper (Figure 5-67) measured the post-failure pillar failure, and there was an overlap period where measurements were made on both cells.

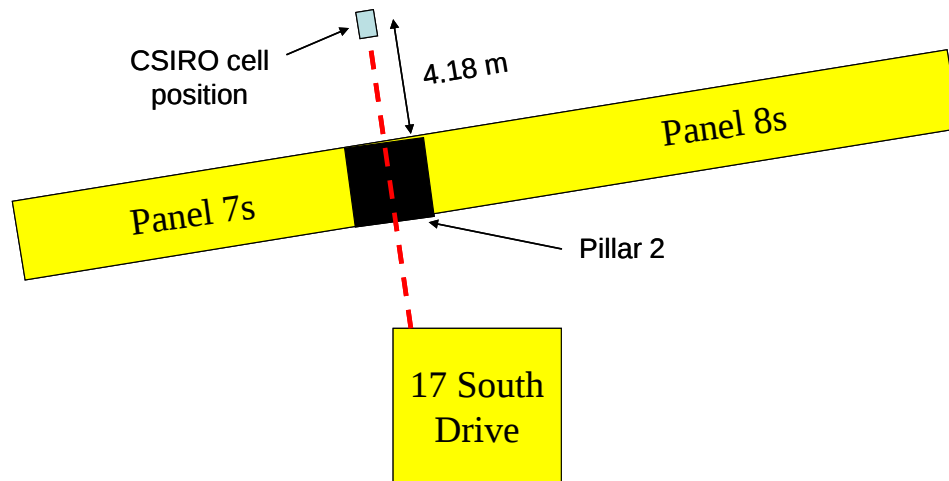


Figure 5-66 Impala site: diagram showing the position of the CSIRO cell above P2 (not drawn to scale)

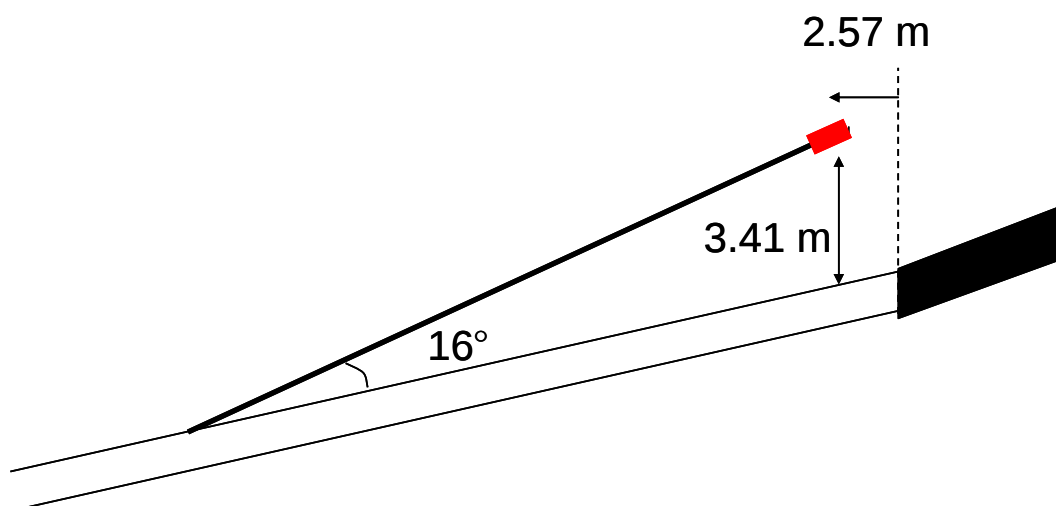


Figure 5-67 Impala site: diagram showing the position of the doorstopper cell above P2 (not drawn to scale)

The measurements over P3 were made 3.23 m above the stope and over the down-dip edge of Pillar 3 as shown in Figure 5-68. The field stress was measured just behind the stress-change cell in the same borehole.

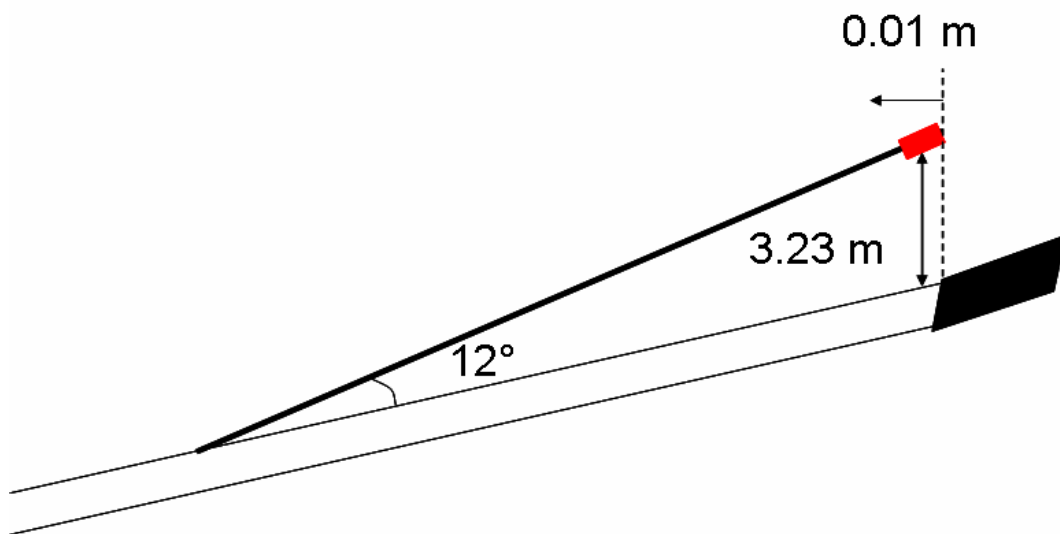


Figure 5-68 Impala site: section showing the position of the doorstopper cell above P3 (not drawn to scale)

The stress change measurements over the remnant pillar (S1) were made 6.66 m above and 1.44 m back from the down-dip pillar edge (Figure 5-69). The field measurement was conducted in the same borehole at the end of the project.

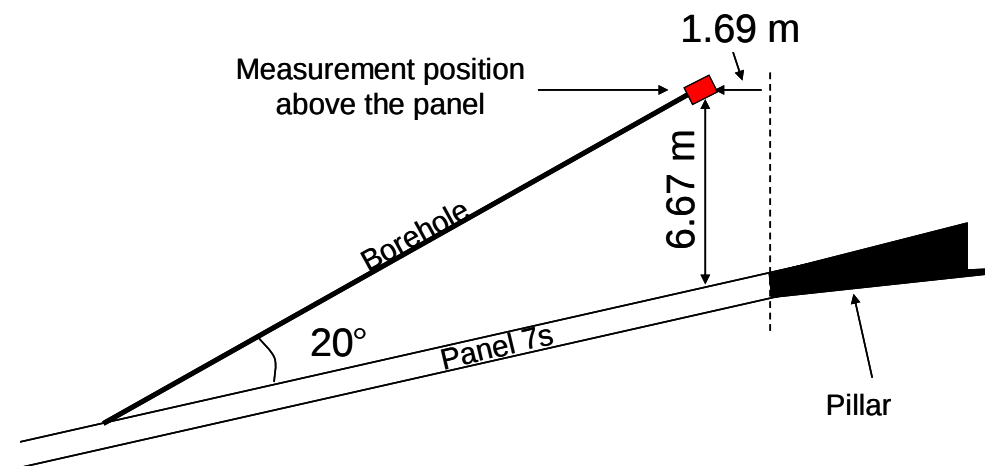


Figure 5-69 Impala site: section showing the position of the doorstopper cell above S1 (not drawn to scale)

5.6.5.3 Impala site: history of pillar formation and failure

Mining operations were temporarily halted in Panel 8s when the face was in line with the centre of P2 (shown by the yellow colour in Figure 5-64). During this time

the ASG was extended beyond P3. Holings were also cut in several places through the strip-pillar between Panels 8s and 7s to the north of P1. In addition, holings were made between P1 and P2, and on both sides of P3, to intersect the existing cubbies defining the down-dip sides of the pillars.

At the time when the 8s face was stopped, the siding was about 9 m behind the face. This meant that the strip-pillar to the north of P1 was very narrow. P1 failed just before the 8s face stopped. P2 appears to have failed when the holing between P1 and P2 was almost completed, and P3 failed during the cutting of the siding.

5.6.5.4 Impala site: stress change measurements

The nominal size of the pillars at the Impala site was 3 m x 6 m in the dip and strike directions, respectively, and the stoping width was about 1.2 m, with a 1.8 m-wide siding between the pillars and the ASG. However, the attention to quality pillar cutting was not good and there was considerable variation in pillar size.

Pillar 1 dimensions were 6.8 m x 5.6 m (length x width) when failure took place. Some of the width was subsequently cut away after failure to form the uneven shape shown in Figure 5-64. The 2D straincell was installed ahead of the up-dip face. Thus stress changes were measured as the stress built up in the abutment and pillar failure occurred about 2.7 m behind the face – i.e. the pillar was fully formed before failure. As a very narrow strip-pillar was cut to the north of P1 (to the right of P1 in Figure 5-64), the effects of the surrounding pillars on the measurement point at failure were minimal. The pillar failed after formation and since the face was relatively far from the measurement point (8 m) there was little interference of the face on the measurements at failure either. The initial readings were converted to stress based on the assumption that the rock mass was linearly elastic in the region where the gauges were applied (Section 3.7.5). Nonlinear behaviour only occurred when the stress at the measurement point dropped to about 6 MPa. The results are plotted against time in Figure 5-70. The methodology used to determine the inferred APS is discussed in Section 5.6.2 and in greater detail in Appendix G. The section of the curve to the right of the

sudden drop in stress (dotted) was evaluated using the techniques developed for nonlinear behaviour (Section 3.8).

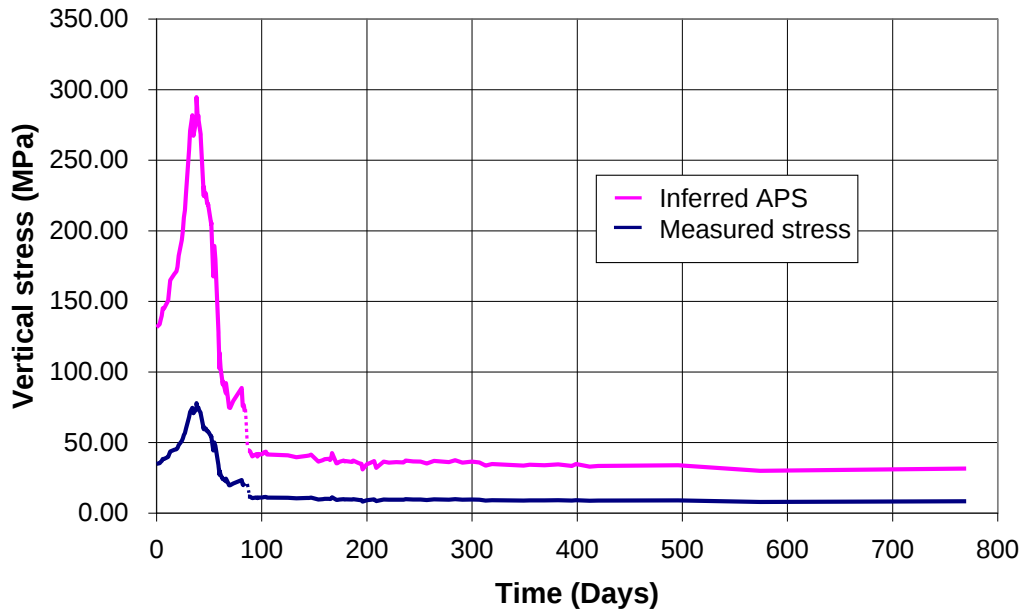


Figure 5-70 Impala site: measurements and inferred P1 stress change plotted against time. The initial pillar stress was determined by a field measurement conducted in a separate, parallel borehole

The P1 analysis suggests pillar peak and residual strengths of 295 MPa and 32 MPa respectively. The peak strength corresponded well to the linear formula described in Section 5.3 (290 MPa) and a MinSim model with the same face positions at pillar failure (296 MPa). These correlations indicate a high level of confidence in the accuracy of the stress measurements. The inferred APS is plotted against strain in Figure 5-71. The strain was calculated from closure measurements conducted in the stope about 1 m down-dip of the pillar. Some inelastic deformations were noted during the closure measurements, particularly when FOGs occurred from shallow-dipping fractures after the pillar had reached its residual stress level. For this reason the deformation used to determine the strain may be exaggerated, particularly beyond about 50 m ϵ .

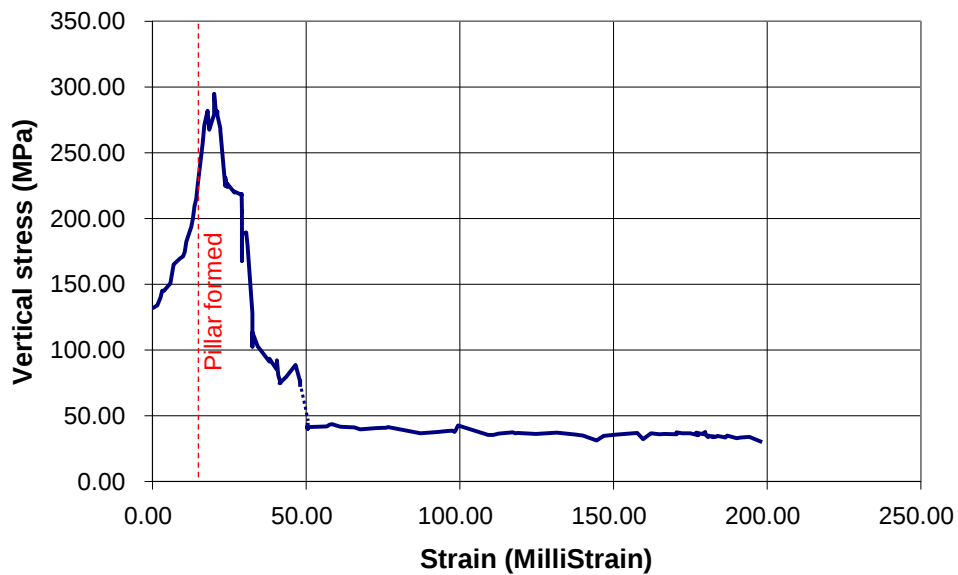


Figure 5-71 Impala site: P1 stress-strain curve estimated from 2D stress measurements conducted at 5.3 m above the pillar and deformation measured in the stope adjacent to the pillar

The up-dip face (Panel 8s) was temporarily stopped at about the mid point of P2 as shown in Figure 5-64. During this time the ASG was extended and holings were cut from the up-dip side to meet the existing cubbies cut on the down-dip side of the pillars. Pillar failure occurred when the holings from the up-dip side were almost holed through into the down-dip cubbies. The dimensions of the pillar at failure were thus similar to P1, but were slightly wider and shorter – i.e. 5.5 m x 6.0 m x 1.2 m (length x width x height). A siding was cut subsequent to failure, which reduced the pillar width to about 3.5 m. Thus the final residual strength was applicable to a 3.5 m-wide pillar. Residual stress levels were reached before the 8s face was advanced.

Two stress-change cells were installed as described in Section 5.6.5.2. The stress results of the 3D cell (P2a) are shown as a function of time in Figure 5-72. This cell was installed above the centre of the pillar from the haulage below as shown in Figure 5-66. As the cell was installed ahead of both the down- and up-dip panels, the full stress history is plotted until just after pillar failure. Unfortunately, the cable used to read the strain gauges was sheared off during pillar failure and no further readings were possible.

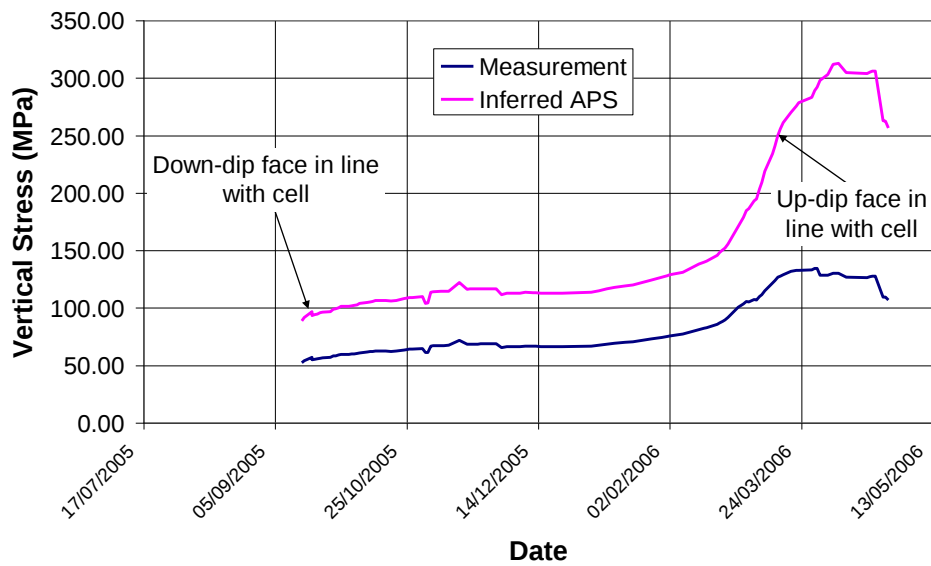


Figure 5-72 Impala site: P2a measurements and inferred APS plotted against time

As the up-dip face (Panel 8s) was stationary for the duration of the pillar failure, the rigorous methodology to determine the pillar strength (as described in Appendix G) was unnecessary. A single conversion factor was derived from a numerical model with the face position and pillar shape at the time of failure. Thus there may be slight errors in the initial and subsequent estimates of APS due to the influence of the up-dip face. However, the pillar formed part of an abutment for most of the monitoring period and therefore could not be properly assessed. A pillar strength of 320 MPa was estimated at failure from the measurements. This is significantly higher than the linear strength formula prediction of 288 MPa. However, it was similar to the APS estimated by MinSim (319 MPa) if it was assumed that S1 failed before P2. (The date that S1 failed was confirmed by the mine overseer's testimony and the seismicity report of the mine.) As the straincell was closer than the recommended distance above the pillar, the results were probably unduly affected by the peak stress at the centre of the pillar. However, this would only have resulted in an error of about 9% (Appendix G). From this analysis it is estimated that the peak pillar strength was somewhere between 291 MPa and 320 MPa (the lower end of the range was not much greater than the calculated strength using the linear formula). An extensometer was also installed through the centre of the pillar and was used to determine pillar strain (Section 4.3). Thus a stress-strain relationship was

established for P2 and plotted in Figure 5-73. As the initial strain was measured when the pillar was part of an abutment, the only useful pillar strain measurements are those that occurred after pillar formation, i.e. above about 270 MPa. The pillar was obviously damaged by this stage as the modulus calculated for this part of the curve is only about 22 GPa.

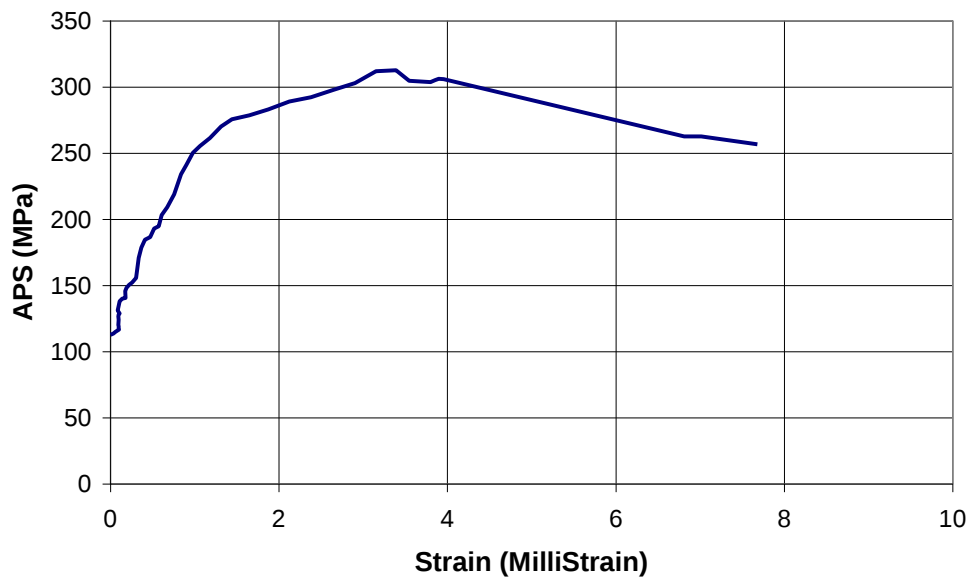


Figure 5-73 Impala site: P2a stress-strain curve using the same APS values shown in Figure 5-72 and an extensometer installed through the centre of the pillar

The results of the 2D straincell located above the centre, but on the down-dip side of P2 (P2b) are plotted against time as shown in Figure 5-74. The position of the cell is described in Section 5.6.5.2. Note that the inferred peak pillar stress of 327 MPa was similar to the inferred peak for P2a, but in this case the absolute stress was adjusted by a field stress measurement performed prior to pillar failure. After pillar failure the inferred APS dropped rapidly to 32 MPa, assisted by the mining of the siding behind the pillar. The stress-strain relationship shown in Figure 5-75 was established by comparing the APS to strain derived from a closure station installed about 2 m below the pillar. The strain recorded between peak and residual for P2b was similar to P1 (approximately 2 m ϵ difference). However, significantly less deformation was measured adjacent to P2 than adjacent to P1 after the pillar had reached residual strength conditions (compare

Figure 5-71 to Figure 5-75). However, the time period over which the measurements were taken was also significantly less.

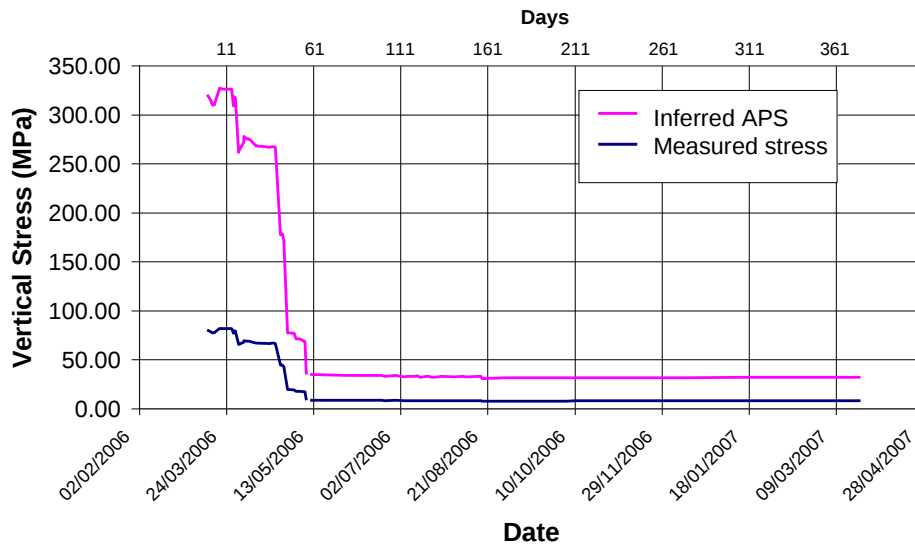


Figure 5-74 Impala site: P2b measurements and inferred APS plotted against time

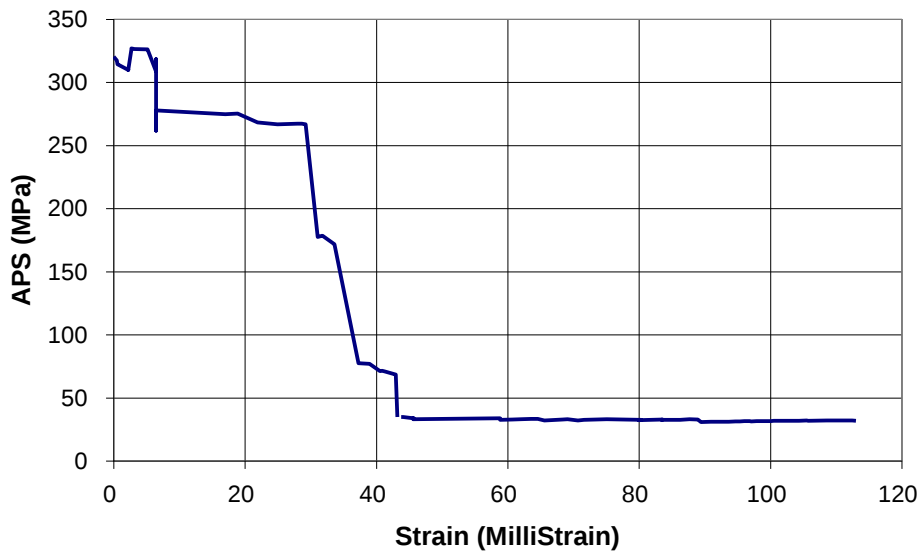


Figure 5-75 Impala site: P2b stress-strain curve using the derived APS values shown in Figure 5-74 and a closure station adjacent to the pillar

P3 was cut ahead of the stationary up-dip face as shown in Figure 5-76. Holings had been cut on both sides of the pillar from the down-dip panel, well before pillar formation. Pillar failure occurred ahead of the face before the pillar was properly formed. Actual measurements of pillar size were not made as sidings were being cut at the time of failure, but a width of about 4 m was estimated. The final pillar width was about 2.5 m. The measurement results and the evaluated APS curve are shown in Figure 5-77. A field measurement was conducted in the same borehole as the stress change measurements, which allowed absolute stress measurements to be determined. A peak and residual APS of 263 MPa and 28 MPa respectively were estimated using the methodology described in Sections 5.6.2 and 5.6.3. The peak compared well to the stress calculated using the linear formula described in Section 5.3 (260 MPa) for pillar dimensions of 3.6 m x 4 m x 1.2 m (length x width x height). The MinSim model, however, suggested a higher stress of 285 MPa.

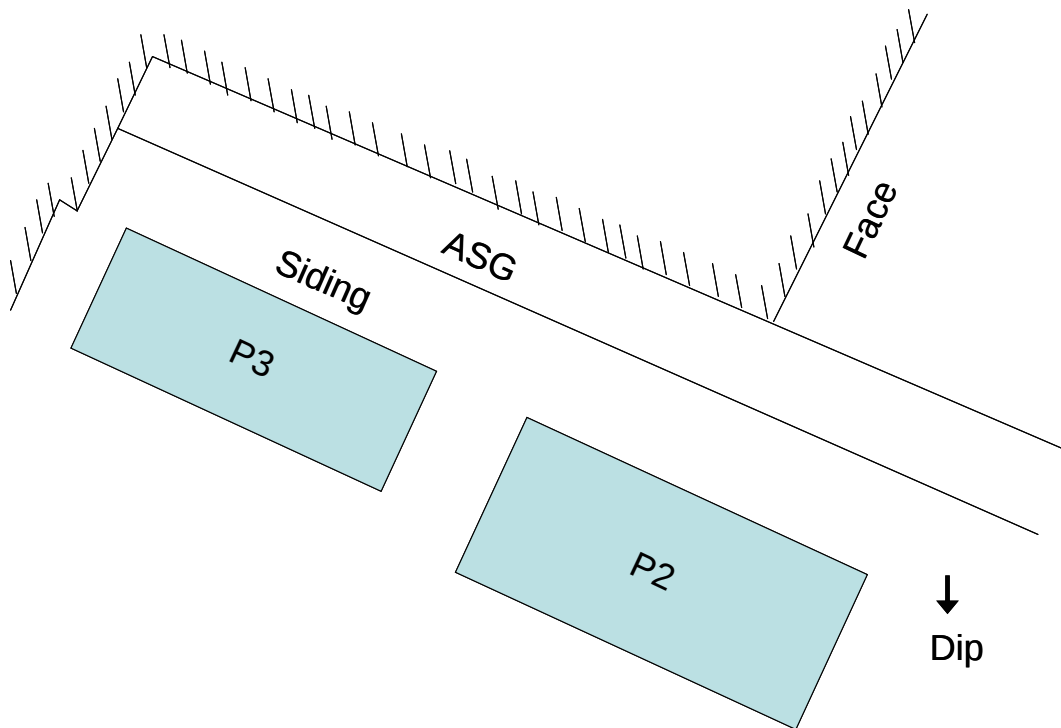


Figure 5-76 Impala site: mining configuration around P3 at failure

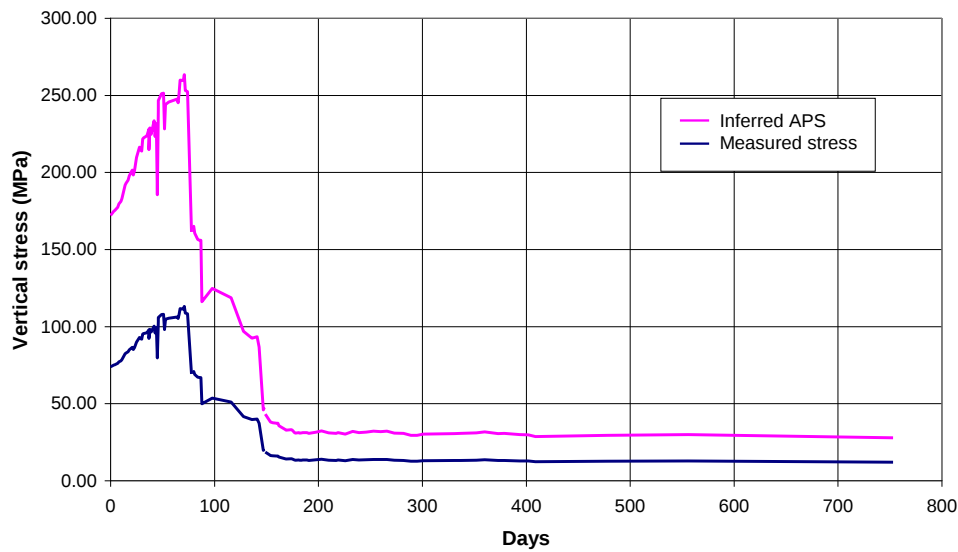


Figure 5-77 Impala site: P3 measurements and inferred APS plotted against time

The estimated pillar behaviour shown in Figure 5-77 was plotted against strain in Figure 5-78. This strain was calculated from closure measurements made immediately adjacent to the down-dip edge of the pillar. The strains recorded between peak and residual were similar to P1 and P2, even though the hangingwall conditions appeared more stable adjacent to this pillar than either of the other pillars. The similar strains shown by the three pillars between peak and residual strength – even though the peak pillar strengths were different and the closure measurements were conducted at different distances from the pillars – suggest that there was little influence of foundation fracturing on the strain results in this region of the curve. However, significantly more strain was measured on P1 than P3 after residual stress levels had been reached, even over a similar time period. This difference in deformation was probably mostly related to foundation fractures and inelastic deformations in the hangingwall adjacent to P1. The hangingwall conditions were significantly more stable adjacent to P3 than P1. No signs of stress regeneration were observed on any of the pillars, at least up to about 100 mE.



Figure 5-78 Impala site: P3 stress-strain curve using the derived APS values shown in Figure 5-77 and a closure station adjacent to the pillar

The loading conditions of P3 at failure were unusually stiff and different from those of the other monitored pillars, as it was cut ahead of the up-dip face (Figure 5-76). This loading environment does not appear to have affected the stress-strain relationship of the post-failure behaviour, but may have influenced the magnitude of the residual strength. Hangingwall conditions around the pillar, and particularly on the down-dip side of the pillar, were significantly more stable than for the other two pillars. This was probably due to the comparatively lower strength of the pillar as it also had smaller dimensions than the other pillars at failure. The relatively high residual strength may also be due to a low stoping width in the siding (0.5 m high in some places), which would have affected the w/h ratio, particularly because the siding appeared more solid than the one in front of P2.

Stress change measurements were made over S1, but unfortunately the cell was installed after pillar failure had taken place. The pillar failed violently and thus the date of failure and face positions was known. A peak APS of 353 MPa was estimated from a MinSim model. This was slightly lower than predicted by the linear equation (363 MPa) for a pillar with dimensions 8 m x 8 m x 1.58 m (length x width x effective height). However, the pillar was triangular and had, therefore,

a variable w/h ratio along its length and the effective width and length are not easily determined. The final stress of 33 MPa was determined from a field stress measurement conducted in the same borehole at the end of the project. This stress condition was surprisingly low as it was similar to the residual strengths of the significantly smaller 'crush' pillars. The very small variation in residual strength between the pillars of different w/h ratios suggests a possible limiting factor on residual strength. It should also be noted, however, that the ASG was cut exceptionally deep (>4 m) adjacent to S1 and this would have influenced the effective w/h ratio of this pillar.

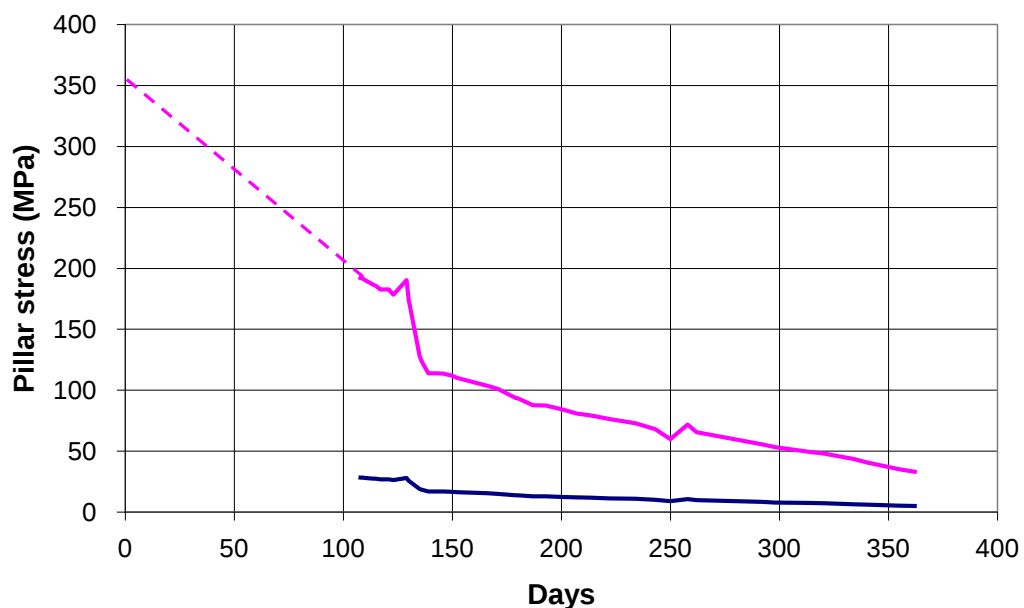


Figure 5-79 Impala site: S1 measurements and inferred APS plotted against time. The peak stress was back-analysed from a MinSim model

No physical differences in the jointing or composition of the pillars were apparent, but the loading conditions were different. P2 was loaded in a stiffer environment than was P1, with half the pillar being cut from the ASG ahead of the up-dip face at the time of failure (Figure 5-76). In addition, P1 had not reached its residual strength and was carrying a significant load at the time when P2 failed. A similar loading condition was true for P3, except that this pillar was cut further ahead of the face than P2 was. P1, however, was loaded in a more uneven manner than the other pillars as there was very little stress on the adjacent strip-pillar at failure.

It was noted from the laboratory tests conducted by Spencer and York (1999), that moderate and severe damage occurred to a depth of almost 1.5 and 0.7 times the original punch size, respectively, at a w/h ratio of 3 (Section 5.4). This depth to size relationship relates to about 7.5 m and 3.5 m below P1. The borehole camera survey described in Section 4.4 suggests that the intensive fracturing was restricted to 2.5 m.

The strain measured in the borehole through the centre of P2 was significantly smaller than the measurements adjacent to the pillar, showing the combined effects of draping and fracturing in the footwall. The comparative magnitude of the closure in the stope signifies a large inelastic component of strain in the stope. The stress strain curves for the three pillars are compared to the strain measurements at the centre of the pillar in Figure 5-80. The three curves determined from the measurements at the edge of the pillars were adjusted so that the peak stresses were in line. Thus a comparison of the post-peak strains could be made. Interestingly, all the pillars reached residual strength between 50 mε and 58 mε. The slightly different post failure path followed by P2 appears to be associated with the reduction in pillar size during pillar formation and time-dependant closure that occurred during a one month period when the up-dip panel was not advanced. The sudden drop in stress was associated with a small seismic event (-0.6 on the Richter scale), which resulted in a visible difference to the shapes of both P1 and P2. The drop in stress in Pillar P1 during the seismic event is shown by the dotted part of the curve. The results of S1 could not be included in the comparison as no strain measurements were made near this pillar.

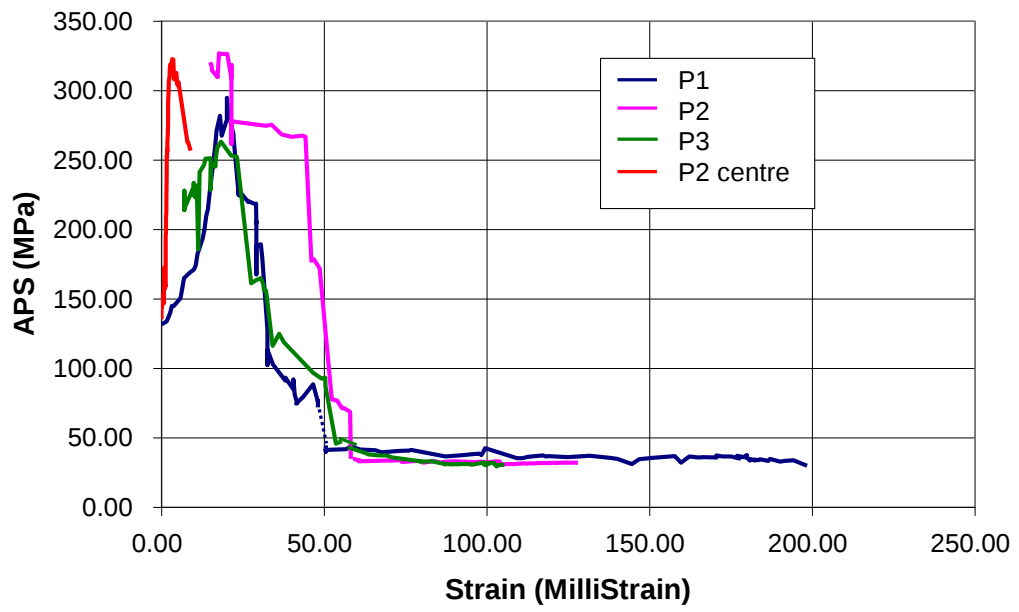


Figure 5-80 Stress-strain curves for the three Impala pillars compared to the strain measurements through the centre of P2

5.6.5.5 Impala site: measurements of residual stress

Once the APS levels on P1, P2 and P3 had dropped to residual strength, 2D and 3D stress measurements were carried out to determine the stress profiles across the pillars and to confirm the residual strengths suggested by the stress change instruments. The measurements were conducted in boreholes drilled between 10° and 15° steeper than the strata (Figure 5-81). As the initial boreholes intersected vertical fracturing at a height of 1.3 m above the pillar, subsequent boreholes were drilled to ensure that the stress measurements were conducted above the vertical fractures.

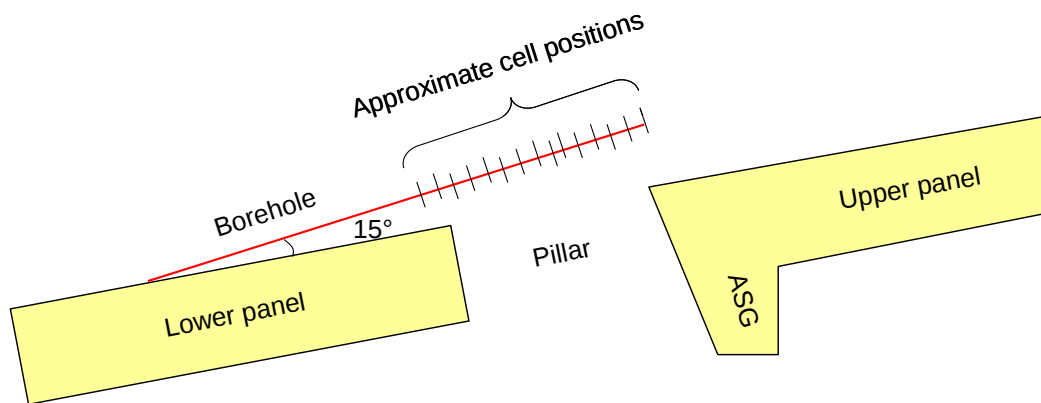


Figure 5-81 Impala site: section showing an example of a borehole used for the pillar residual stress measurements (not drawn to scale)

The measurements over P1 were conducted in a shallow-dipping borehole (Figure 5-81) at heights of between 1.9 m and 3.4 m above the mid-line of the pillar. The direction of the drilling is shown by the arrows in Figure 5-64. A more detailed description of the measurement boreholes is provided in Appendix G.

The results of the P1 stress measurements are shown in Table 5-11. A low error was determined for most of the results indicating a good data set. (Errors of up to 10% are considered acceptable.) Measurements were conducted over the 5 m-wide pillar and over the stope on either side of the pillar.

Table 5-11 Impala site: stress profile measurements across Impala P1 from borehole P1a. The measurements conducted over the pillar are shaded in yellow

Distance from pillar edge (m)	Height above pillar (m)	Error in strain measure. (%)	Vertical stress (MPa)
-0.31	1.93	3.4	11.5
0.06	2.03	1	13.1
0.48	2.15	1.2	17.7
0.82	2.24	3.8	19.5
1.20	2.35	12.9	21.8
1.98	2.56	1.5	30.0
2.33	2.66	2.8	21.8
2.58	2.73	6.9	20.0
2.91	2.82	4.8	17.8
3.28	2.92	2.6	18.3
3.64	3.02	1.8	15.5
4.09	3.15	9.5	14.5
4.49	3.26	4.1	12.1
4.79	3.34	37.9	9.4
5.17	3.44	38.1	8.6

The vertical 'stress-fractures' that were observed up to about 1.3 m above the pillar might have affected the stress distribution. In addition, stress evaluations in the nonlinear rock meant that the strain-to-stress conversions were uncertain as further work is needed to confirm or improve the methodology adopted (Section 3.8). 'Joint-channelling' of stress further complicated the analysis. The measured profile in Figure 5-82 should, therefore, be considered as a rough guide to the stress profile and could not be replicated using a reasonable inverse matrix of Boussinesq equations. A 'best-fit' solution (as described in Section 5.6.3) was therefore used to determine the pillar stress profile. The 'best-fit' solution and the resultant pillar stress profile are included in the figure. The evaluation methodology is discussed in greater detail in Appendix G.

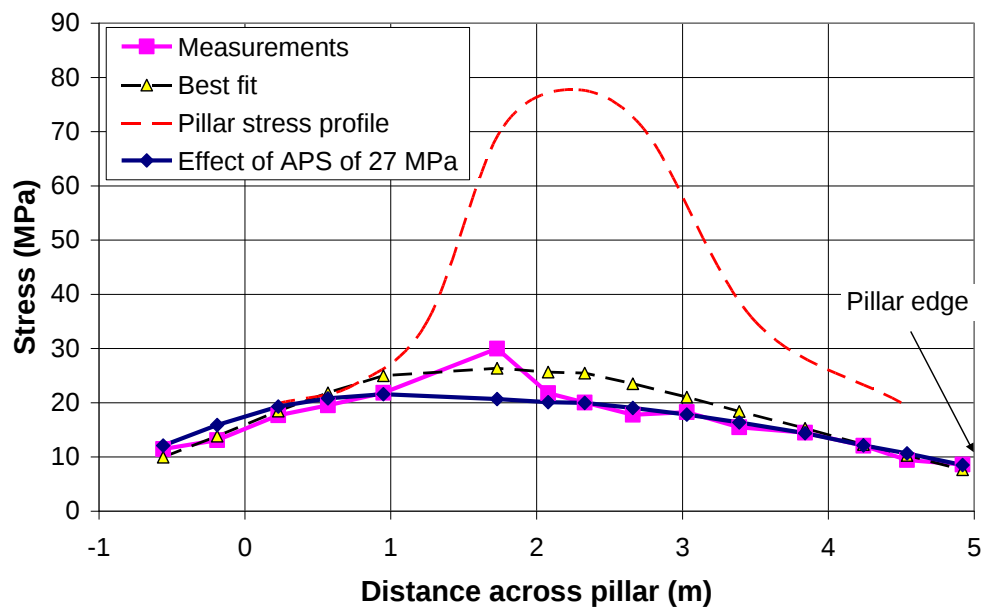


Figure 5-82 Impala site: comparison between the measured, uniform (APS = 27 MPa), ‘best-fit’ and back-calculated stress profiles across P1

An almost flat distribution of stress with a slight peak towards the centre was suggested by the good correlation between the measurements and the uniform stress distribution. An APS of 27 MPa was suggested by this analysis (Figure 5-82). Since the measurement that indicated a peak near the pillar centre had a low correlation error, a ‘best-fit’ curve was required to take cognisance of it. The final pillar stress evaluation suggested a peak stress of about 78 MPa near the centre of the pillar and an average residual strength of about 27 MPa, which was slightly lower than the stress change measurements (32 MPa) in Figure 5-70. The punch tests conducted by Spencer and York (1999) suggest a residual strength of 29 MPa for a pillar of this w/h ratio.

Figure 5-83 shows the stress distribution across the pillar resulting from the ‘best-fit’ analysis in 3D. The smooth profile with the peak at the centre suggests that failure had occurred throughout the pillar system. A comparison between this profile and the Wagner (1980) measurements (Figure 5-7) suggests a stable condition, which was confirmed by the stress-change measurements given in Figure 5-70.

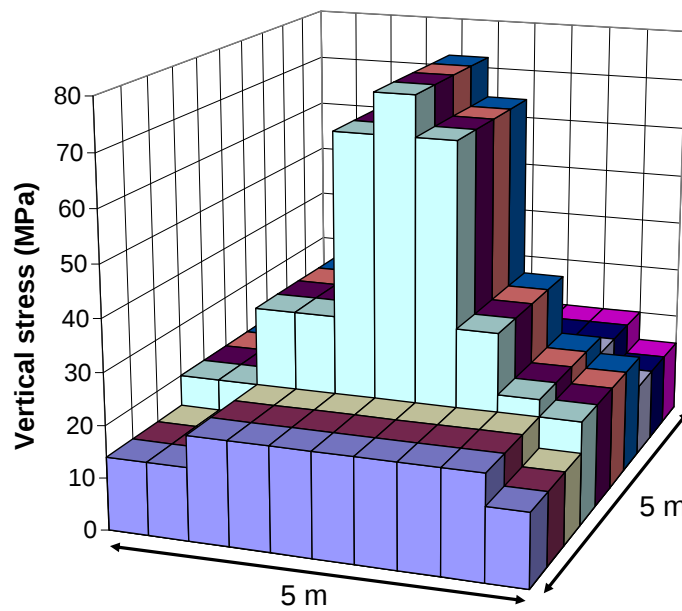


Figure 5-83 Impala site: 3D stress profile across P1 (derived from a Boussinesq inverse matrix)

Stress measurements were conducted in three boreholes over P2. Two of these boreholes (P2a and P2b) were parallel and collared about the same distance from the pillar. The results for these two data sets are provided in Table 5-12 and Table 5-13, respectively, and were evaluated together in Figure 5-84. The comparatively high measurement at 1.39 m from the pillar edge in P2b (Table 5-13) was measured adjacent to a vertical joint. This again suggests the concept of 'joint-channelling'. The 'best-fit' curve therefore was estimated between the high and low stress measurements conducted in the P1a and P1b boreholes. The pillar profile in Figure 5-84 was based on the 'best-fit' curve. The evaluation suggests a peak stress of 94 MPa near the centre of the pillar and a residual strength of about 21 MPa. This evaluation provides a substantially lower residual APS than the stress change measurements (32 MPa in Figure 5-74).

Table 5-12 Impala site: stress profile measurements across P2 – from borehole P2a. The measurements over the pillar are shaded in yellow

Distance from pillar edge (m)	Height above pillar (m)	Error in strain measure. (%)	Vertical stress (MPa)
0.21	1.28	5.1	10.14
1.31	1.47	2.1	24.98
1.65	1.53	5.8	19.72
1.95	1.58	1.4	15.33
2.24	1.63	0.4	14.44
2.68	1.71	1.2	10.91
2.93	1.75	3	9.86
3.3	1.81	3	10.25
5.08	2.12	8.8	-0.01

Table 5-13 Impala site: stress profile measurements across P2 – from borehole P2b. The measurements over the pillar are shaded in yellow

Distance from pillar edge (m)	Height above pillar (m)	Error in strain measure. (%)	Vertical stress (MPa)
-0.77	1.27	6.3	5.2
0.08	1.42	8.2	9.2
0.29	1.46	5.1	14.9
0.61	1.52	-	20.3
1.39	1.65	4	39.7
2.22	1.80	10.4	20.5
2.59	1.87	2.9	14.5

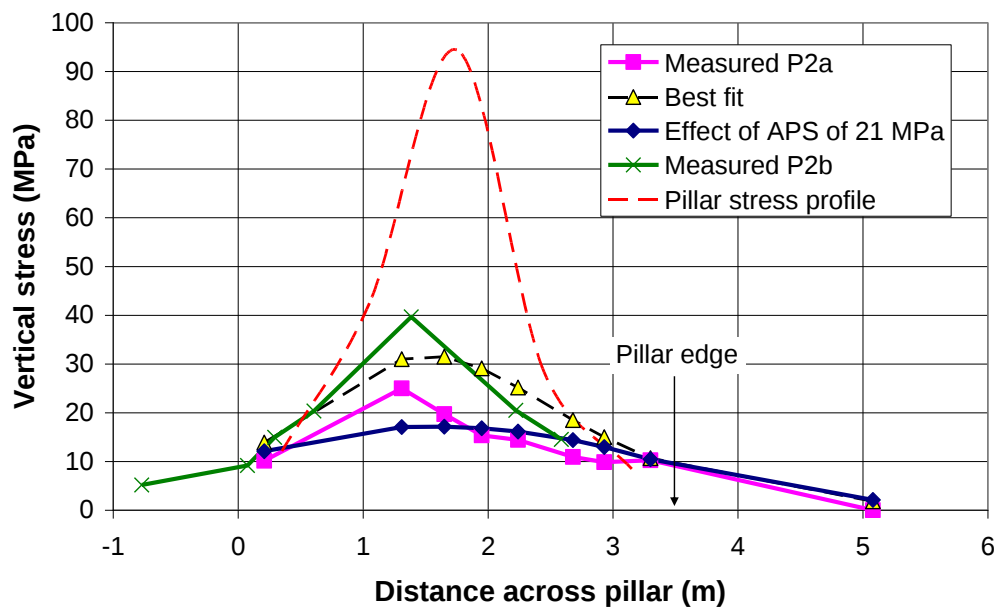


Figure 5-84 Impala site: comparison between the measured, uniform APS of 21 MPa, ‘best-fit’ and back-calculated stress profiles across P2 – using the measurements in Boreholes P2a and P2b

The assumptions about the stress level of the ‘best-fit’ curve, due to ‘joint-channelling’, may have resulted in an underestimation of stress if the highest peaks were not measured. The very high peak might also have been exaggerated because of the inability of the elastic Boussinesq evaluations to account for ‘joint-channelling’. A third set of stress measurements was therefore conducted at heights of between 2.1 m and 3.4 m above the pillar to reduce the ‘joint-channelling’ effects. These measurements were conducted using triaxial cells and are shown in Table 5-14. The measurements and the inverse matrix evaluation of the ‘best-fit’ curve to these measurements are shown in Figure 5-85. The solution provided a more gentle stress profile across the pillar than in Figure 5-84 but with a residual strength of 32 MPa, which is equivalent to the stress change measurements in Figure 5-75. The latter investigation suggests a peak of 70 MPa at the centre of the pillar. A back analysis of this curve suggested similar stresses to those that were measured at about 4 m above the pillar. A comparison between the back-calculated and measured stresses is provided in Table 5-15.

Table 5-14 Impala site: stress profile measurements across P2 from borehole P2c. The measurements over the pillar are shaded in yellow

Distance from pillar edge (m)	Height above pillar (m)	Error in strain measure. (%)	Vertical stress (MPa)
-0.88	2.10	7.2	8.6
-0.59	2.17	15.0	7.6
-0.06	2.31	9.3	14.4
1.73	2.80	8.7	16.2
2.21	2.92	7.4	7.3
2.94	3.12	8.9	10.3
3.30	3.21	7.6	10.6
3.88	3.37	4.2	10.2

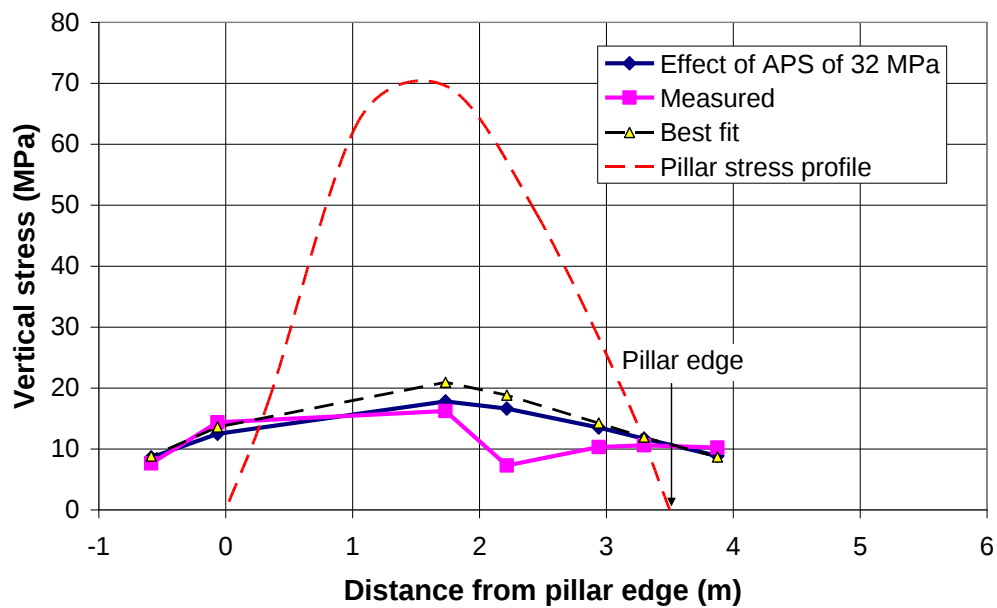


Figure 5-85 Impala site: comparison between the measured, uniform (APS = 32 MPa), 'best-fit' and back-calculated stress profiles across P2 - using Borehole P2c

Table 5-15 Impala site: comparison between the measured and back-analysed stresses in P2b_f

Height above pillar (m)	Measured stress (MPa)	Error in strain measurement (%)	Back analysed stress (MPa)
3.7	9.27	14.3	8.29
3.9	10.23	0.7	10.35

The good correlation between the back-analysed results shown in Figure 5-85 and the measurements at greater height in Table 5-15 suggests that the pillar stress curve seen in Figure 5-85 may be more representative of the actual stress profile across P2 than that shown in Figure 5-84. The very good correlation between the uniform stress profile and the measurements in Figure 5-85 also suggests that there may not be a high peak stress at the centre of the pillar. However, no measurements were made across the initial half of the pillar as the borehole intersected a joint that was oblique to the borehole. For this reason the measurements that may have defined the peak stress at the centre of the pillar were not made.

The 3D stress profile derived from the available measurements is shown in Figure 5-86. The evaluations suggest that the residual strength of Pillar P2 is similar to that of P1. Since P2 is significantly smaller than P1, the residual strength is expected to be lower. However, P2 was confined by a siding and the gully adjacent to P1 created a high sidewall on one side of that pillar. The fractured rock in the high sidewall may not have provided much more confinement to the centre of the pillar than the siding adjacent to P2.

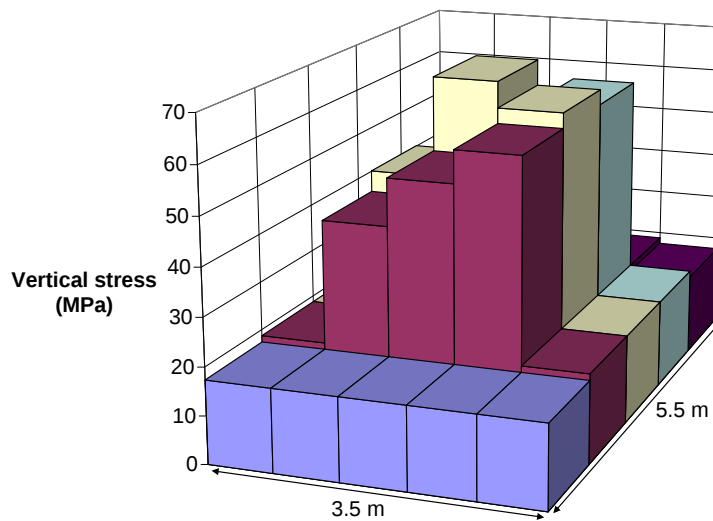


Figure 5-86 Impala site: stress profile across P2 (derived by the Boussinesq inverse matrix)

The residual strength of P3 was determined by stress measurements conducted in two boreholes as shown in Figure 5-87. In plan the boreholes were drilled across the narrow section of the pillar at the centre of the long axis (Figure 5-88). Nine biaxial straincells (doorstoppers) and one CSIR triaxial cell were installed in the shallow-dipping borehole (15° steeper than the reef). This borehole was drilled from the down-dip panel (7s). Two CSIR triaxial cells measured the 3D stress conditions from a steeply dipping borehole drilled from the up-dip panel. The results are provided in Table 5-16 and Table 5-17. Three high stresses were measured in the shallow-dipping hole between 1.13 m and 2.44 m from the edge of the pillar (Table 5-16). These measurements were made between two vertical joints, again suggesting 'joint-channelling' as shown by the Elfen modelling in Figure 5-60. The resolution of these stresses into a pillar profile by using the normal inverse matrix evaluation (Section 5.6.3) provided a very high average residual strength of 120 MPa. Since the hangingwall conditions were stable and excessive closure adjacent to the pillar was not measured, high stresses seem inconsistent and 'joint-channelling' is apparent. The value of the highest stress in Table 5-16 was even higher than the peak stresses evaluated for the other two wider pillars (P1 and P2), again suggesting 'joint-channelling'. A Boussinesq matrix was used to evaluate the stress measurements in the shallow-dipping borehole that were conducted outside of the joints (Figure 5-89). The stress levels at the positions of the high stresses were interpolated assuming the

stresses between the joints had been approximately ‘channelled’ up to the elevation of the measurements – i.e. the measured stress was the actual pillar stress in the region shown by the green shading in Table 5-16. This assumption resulted in a similar average residual strength to that measured on the stress-change cell (28 MPa). The peak stress near the centre of the pillar was about 88 MPa, which was similar to the initial P2 evaluation for the two data sets in Figure 5-84. Since the evaluated P3 stress profile is the lowest possible profile for the given set of measurements – i.e. the pillar stresses are the same as the measurements, the relatively high predicted peak stress seems reasonable.

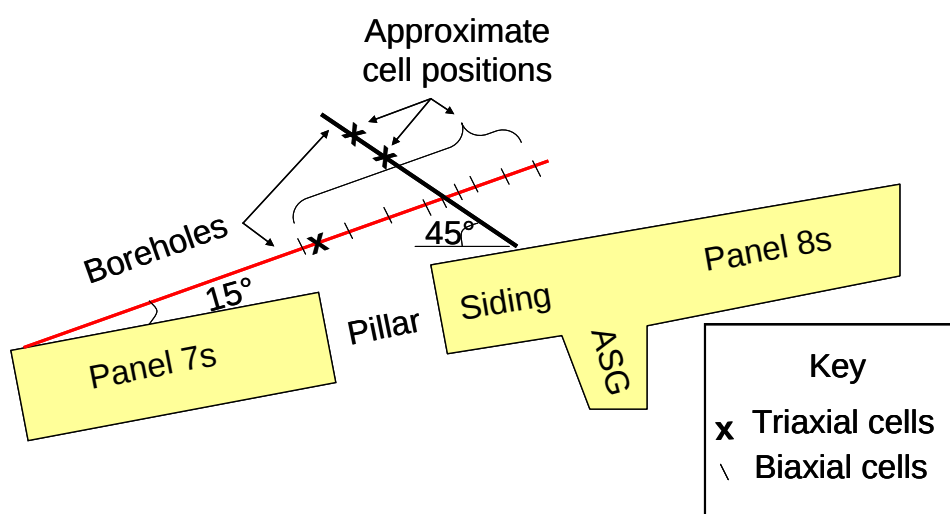


Figure 5-87 Impala site: section showing the instrumentation positions above P3 (not drawn to scale)

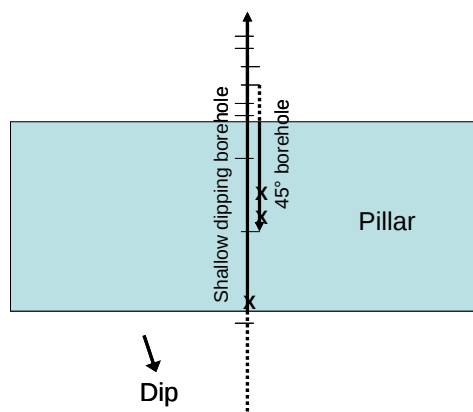


Figure 5-88 Impala site: plan view showing the instrumentation positions above P3 (not drawn to scale). X = triaxial cells, — = biaxial cells

Table 5-16 Impala site: stress-profile measurements across P3 from Borehole P3a. The green shading highlights the measurements considered to be 'channelled' and the yellow and green shading show the measurements conducted above the pillar

Distance from pillar edge (m)	Height above pillar (m)	Error in strain measure. (%)	Vertical stress (MPa)
-1.07	1.87	6.8	10.0
-0.77	1.95	-	22.0
0.44	2.27	9.3	11.1
1.13	2.46	5.7	89.4
1.91	2.67	1.3	55.2
2.44	2.81	-	31.0
4.15	3.27	5.1	3.6
4.49	3.36	8.2	-0.3
5.40	3.60	6.1	0.4

Table 5-17 Impala site: stress-profile measurements above P3 from Borehole P3b. The yellow shading shows the measurement conducted above the pillar

Distance from pillar edge (m)	Height above pillar (m)	Error in strain measure. (%)	Vertical stress (MPa)
2.56	3.74	8.2	36.2
2.35	4.05	7.7	26.7

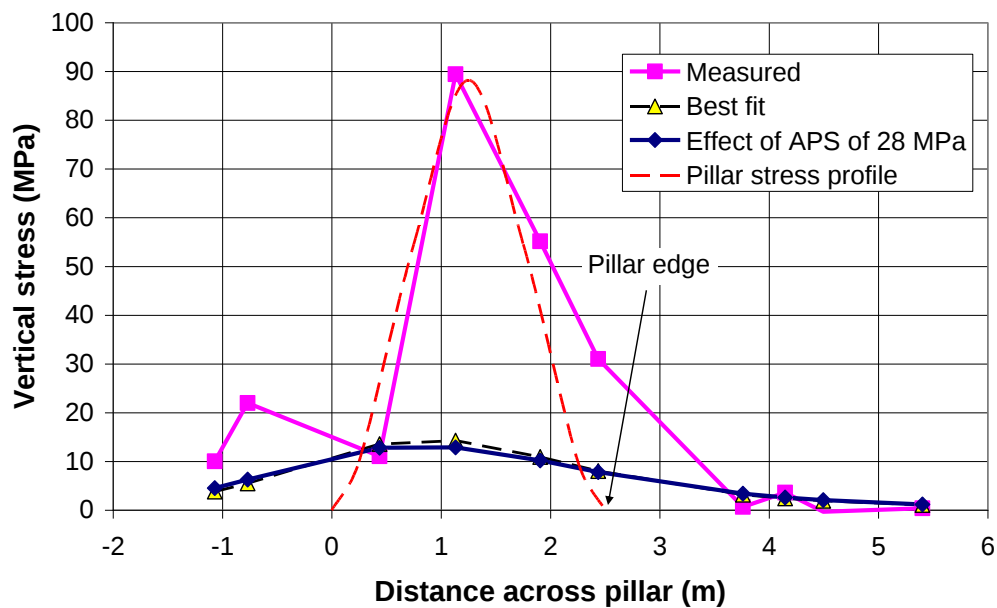


Figure 5-89 Impala site: comparison between the measured, uniform (APS = 28 MPa), 'best-fit' and back-calculated stress profiles across P3 – from Borehole P3a

The assumption of 'joint-channelling' that was used to evaluate the measurements between the joints in the shallow-dipping borehole was also applied to the measurements in the steeply dipping borehole (Table 5-18) – i.e. the heights of the measurements were moved about 2.5 m closer to the pillar. The back-analysed results of the pillar stress profile in Figure 5-89 are compared to the measurements in Table 5-18. The comparison shows a reasonably good match between the measurements and the back-analysed stresses.

Table 5-18 Impala site: comparison between the measured and back-analysed stresses from Borehole P3b

Height above pillar (m)	Measured (MPa)	Back-analysed (MPa)	Difference %
3.74	36.2	32.6	10
4.05	26.7	25.6	4

The 3D stress profile estimated from the analysis shown in Figure 5-89 is provided in Figure 5-90. The comparatively high stress conditions towards the centre of the pillar may be the result of the small stopping width in the siding (0.5 m in some places). The pillar also failed in a very stiff and controlled environment (ahead of the face), which was not the same as for P1 and P2.

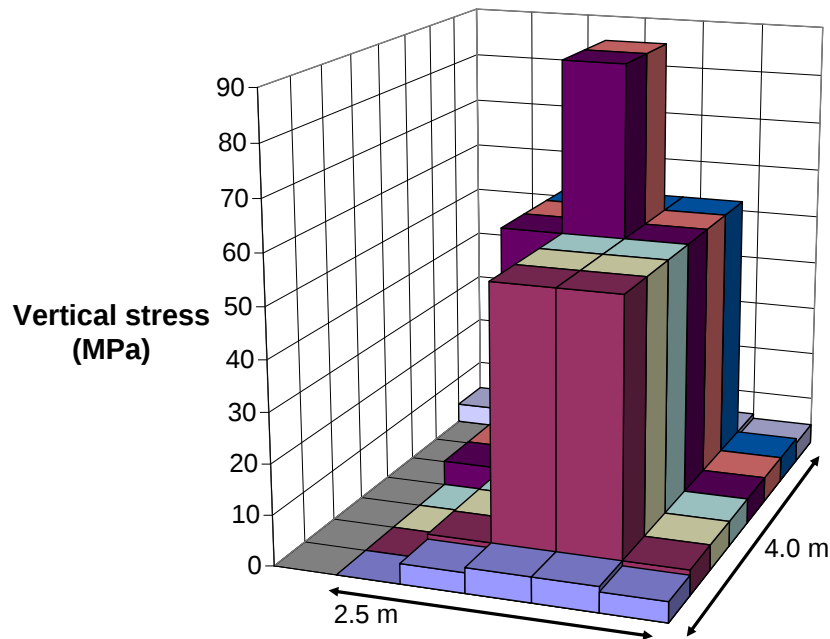


Figure 5-90 Impala site: stress profile across P3 (determined from the Boussinesq inverse matrix)

The range of pillar residual strengths was remarkably small, considering the range of w/h ratios (Figure 5-91). Even though the various loading conditions and pillar shapes and sizes probably influenced the results, the strength range is not adequately explained. The results suggest the influence of an overriding factor that overwhelms the effects of w/h ratio on residual strength. One such factor could be the result of pillar punching into a damaged footwall. Underground measurements of pillar behaviour performed by Lougher (1994) and Spencer and York (1999) at Impala Platinum Mine positively linked the influence of a strata-parallel discontinuity in the footwall to residual strength. However, in both instances, the residual strength was significantly higher than those measured at this site and indicated some form of 'yielding' pillar.

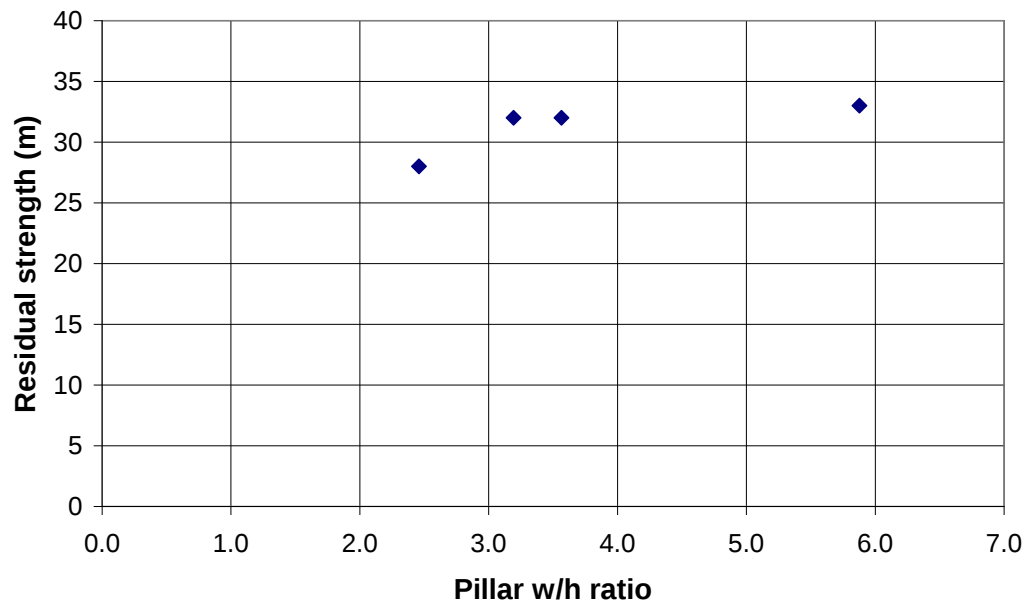


Figure 5-91 Impala site: pillar residual strength as a function of w/h ratio

5.6.6 Intermediate depth poor rock mass conditions (Union site)

5.6.6.1 Union site description

The measurements were conducted at Union Section, Spud Shaft in Stope 27-31b-3s at a depth of about 1400 m below surface. The rock mass conditions are described as poor (Section 2.3), with numerous shallow- and steeply dipping joint sets. An isolated pillar was cut at the centre of the panel for monitoring purposes. Although the orientation of the pillar with its long axis on dip was abnormal, the site provided an excellent opportunity to measure the stress change over a pillar without the influence of adjacent pillars. The instrumentation site is shown as 3s in Figure 5-92 and indicated as “Site 1” in Chapter 4. In the figure, face advances are indicated by an array of colours while the mining prior to the instrumentation installation is shown in red. The grey represents unmined areas and pillars.

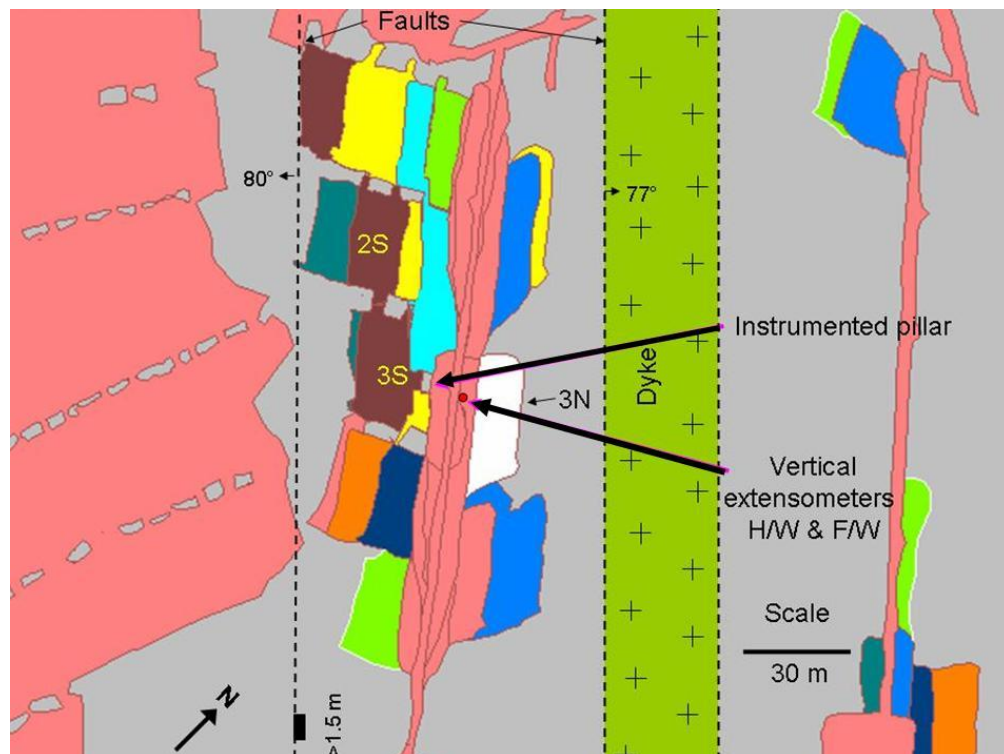


Figure 5-92 Union site: stope sheet showing the location of the instrumented pillar, with mining steps represented by different colours. Stope on left mined prior to instrumentation installation

As the instrumented pillar was additional to the normal pillar configuration, it was cut to smaller than standard dimensions to determine the feasibility of using such dimensions in the future. The important issues were: the peak strength, as the current pillars often damage the hangingwall in poor rock mass conditions; and the residual strength during the working life of the stope, as the pillars must prevent parting on the Bastard Reef some 20 m to 25 m above.

5.6.6.2 Union site: instrument locations

2D straincells were installed above the proposed position of the isolated pillar prior to pillar formation. A stope sheet showing the face position and the future position of the isolated pillar is provided in Figure 5-93. Straincells were installed in two shallow-dipping boreholes at 3.4 m (P1a) and 5.4 m (P1b) above the abutment at the edge of a ledged raise, respectively. A sectional view of the cells in relation to the proposed pillar is shown Figure 5-94.

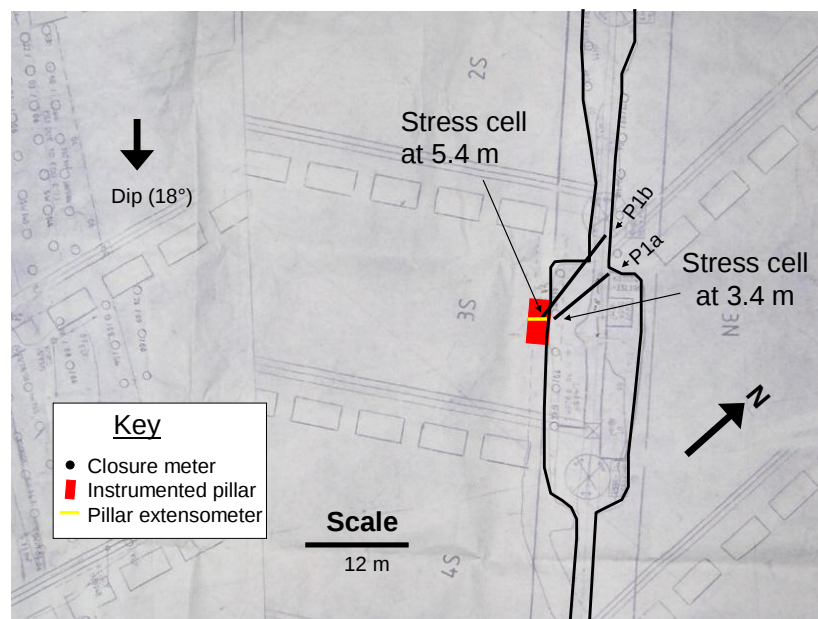


Figure 5-93 Union site: stope sheet showing the ledged centre-raise and proposed position of P1

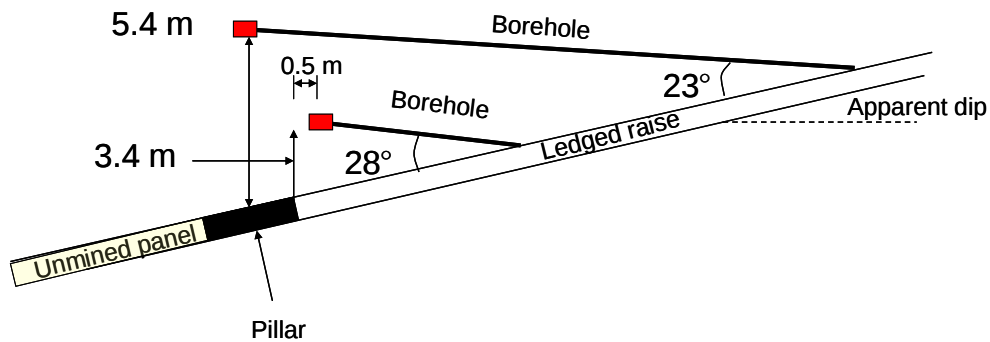


Figure 5-94 Union site: section showing the straincell locations (P1a and P1b) relative to the pillar (not drawn to scale)

The initial cells (Figure 5-94) were vandalised soon after pillar failure and a new (3D) cell was installed in a steeply dipping borehole after mining had been completed in Panel 3s (P1c in Figure 5-95). This cell was installed on the south side of the pillar at a height of 6.45 m above the centre of the pillar and 1 m in from the pillar edge in plan. A stope sheet showing the mining configuration during and after the straincell installation is provided in Figure 5-96. The mining configuration at installation is shown in red and the array of colours indicates face advances that occurred subsequent to the cell installation in the figure. Measurements initiated at this cell location on 6 September 2007.

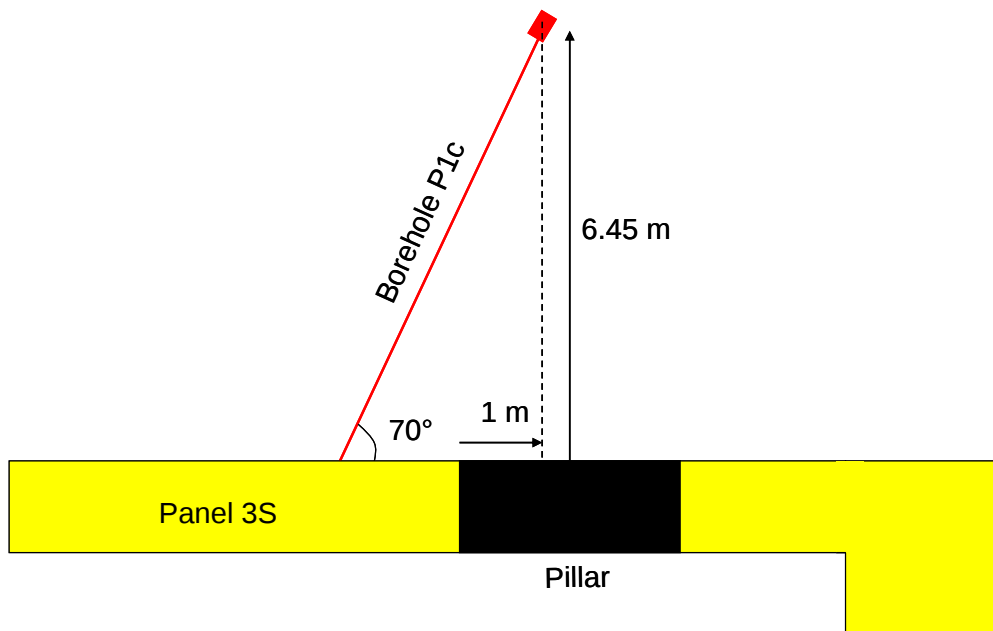


Figure 5-95 Union site: section showing Borehole P1c over P1 (not drawn to scale)

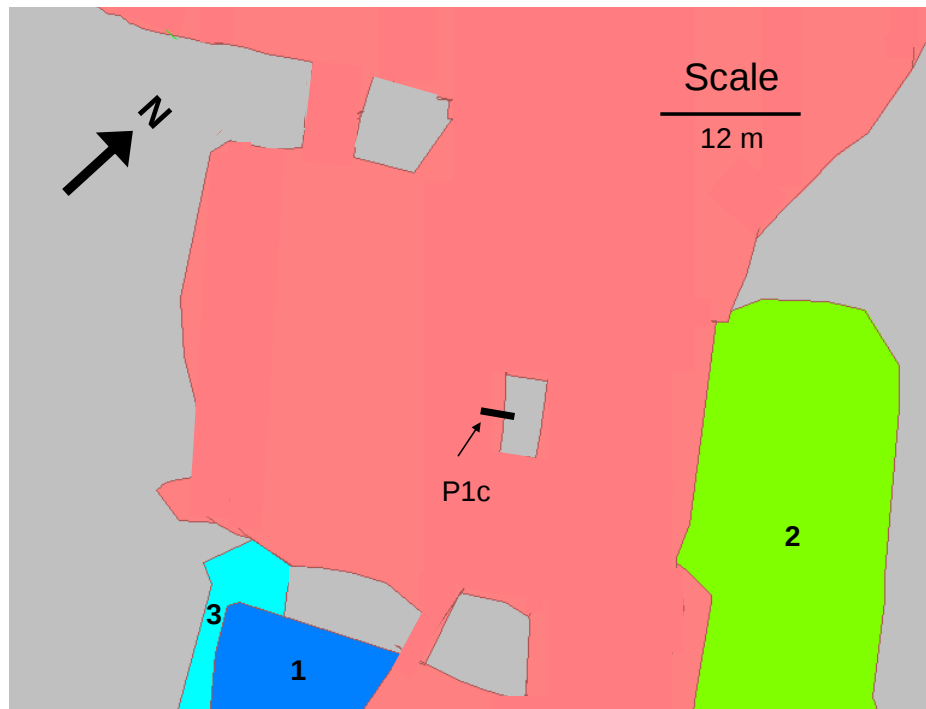


Figure 5-96 Union site: stope sheet showing the approximate position of the P1c stress-change cell. Mining steps after installation of P1c are represented by the array of colours

5.6.6.3 Union site: history of pillar formation

The face on the up-dip side of the proposed isolated pillar was mined on breast to the position shown in Figure 5-97 prior to any advances of the face on the down-dip side of this pillar. Subsequently, the down-dip face was advanced to be in line with the up-dip face and a wide-end was advanced up the back of the proposed pillar from the down-dip side. Pillar failure occurred when the wide end had advanced up-dip approximately 2.2 m (Figure 5-97). After the wide end holed from the bottom into the mining on the up-dip side, the whole face length was advanced on breast between the proposed pillars defining Panel 3s (Figure 5-97). The final face position is where mining was terminated in the panel due to poor ground conditions.

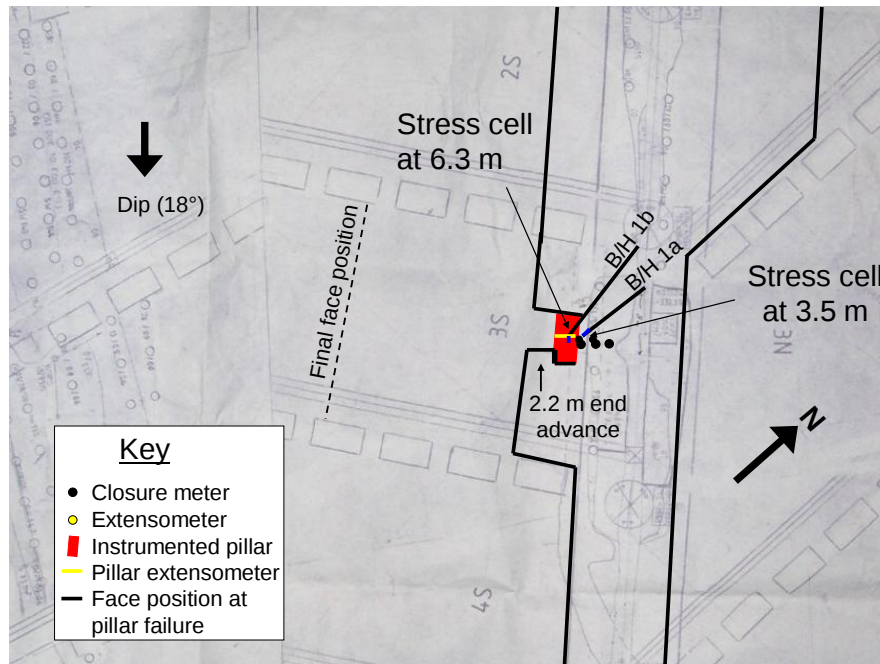


Figure 5-97 Union site: stope sheet showing the face position at pillar failure

5.6.6.4 Union site: pillar description

Stoping widths in the ledge – i.e. the north side of the pillar – were fairly uniform, ranging between 1.2 m and 1.4 m, but mainly 1.4 m. On the south side of the pillar, the stoping width was less consistent, with a range of heights between 1.2 m and 1.8 m. The final pillar dimensions were 2.8 m on strike and 5.3 m on dip with an average height of about 1.5 m – i.e. a w/h ratio of 1.9.

5.6.6.5 Union site: stress change measurements

Field stress measurements were conducted just behind the stress-change cells at each of the two measurement positions. Thus the stress-change measurements were adjusted to show the absolute stresses. The results of the stress-change measurements conducted in boreholes P1b and P1c were combined into a single graph and used to evaluate the pillar stress changes during the pillar evolution. The results of the investigation are shown together with the pre-failure elastic

(MinSim) pillar stresses in Figure 5-98. A reasonably good correlation was shown between the elastic model and the measurements prior to pillar failure.

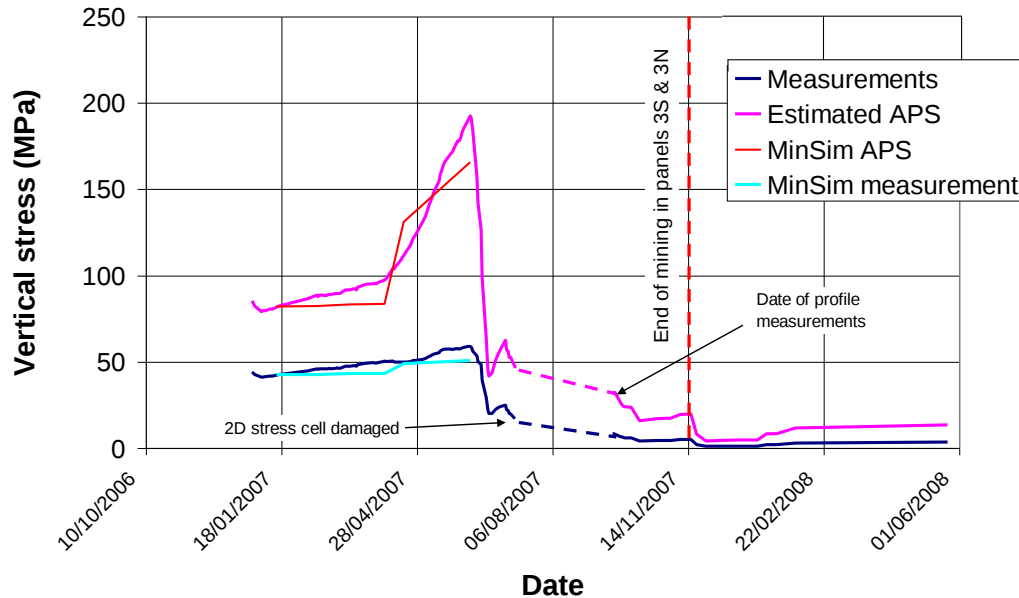


Figure 5-98 Union site: vertical stress measurements and estimated P1 APS and the results of an elastic MinSim model

The investigations suggest a pillar peak stress of 191 MPa, which was about 25 MPa higher than the elastic model. The linear pillar strength equation (Section 5.3) suggested a slightly higher strength of 207 MPa if the pillar dimensions of 5.3 m x 2.8 m and a height of 1.5 m were assumed. However, if the pillar height was assumed to be 1.8 m in the equation, the equation result was similar to the measurements. It should be noted that the pillar was not properly formed when pillar failure occurred, which means that the effective pillar width at failure is not clearly defined. The geomechanical conditions at the Union site were also different from the Impala sites used in the development of the formula – i.e. in the strength back-analysis described in Section 5.3. For instance, the footwall at the Union site was weaker than at the Impala sites (Figure 5-99) and probably affected the pillar strength.

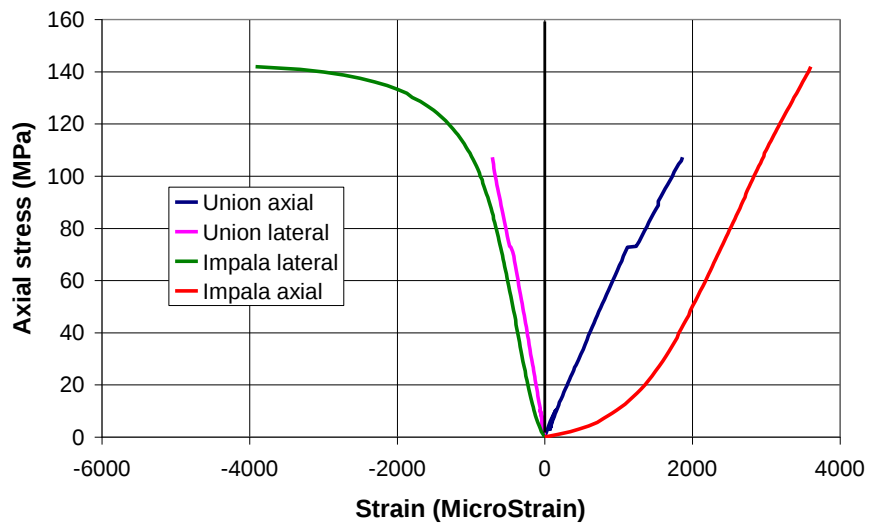


Figure 5-99 Stress-strain curves for the footwall material immediately below the pillars at the Union and Impala sites

The results that can be seen in Figure 5-98 show that the residual strength dropped to about a minimum of 16 MPa during stoping operations. However, once mining had terminated in the stope a further drop in stress to about 4 MPa was observed. This drop was followed by a small stress increase to 14 MPa over a period of about five months. Significant fracturing was observed on the north edge of the pillar during failure as shown by the comparison presented in Figure 5-100.

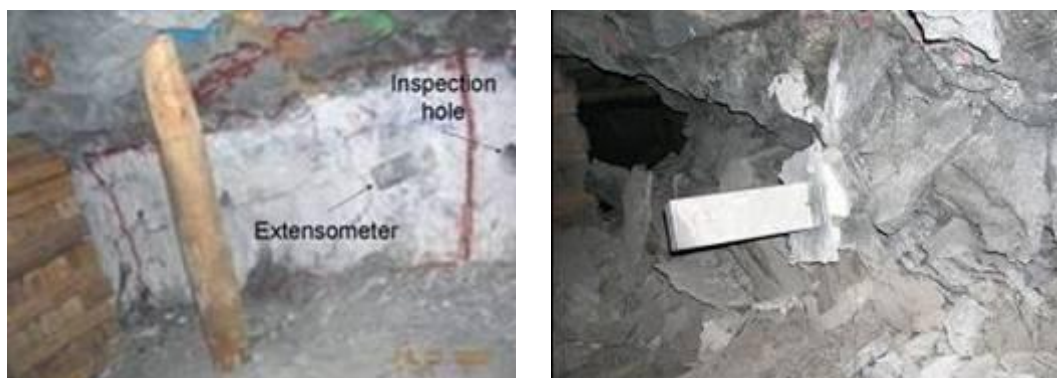


Figure 5-100 Union site: north side of P1 before and after pillar formation

In contrast to the fracturing on the north side of the pillar, the south side was almost unfractured at the time of developing the wide end (Figure 5-101) The

typical fractures expected of a failed pillar were also not observed in the sidewall or hangingwall on this side of the pillar. This lack of fracturing suggests that the pillar failure may have been restricted to about 2.8 m on the north side of the pillar stub and that the wide end was excavated in ‘solid’ rock conditions. Thus the final pillar dimensions may have been the approximate size of the pillar at failure.

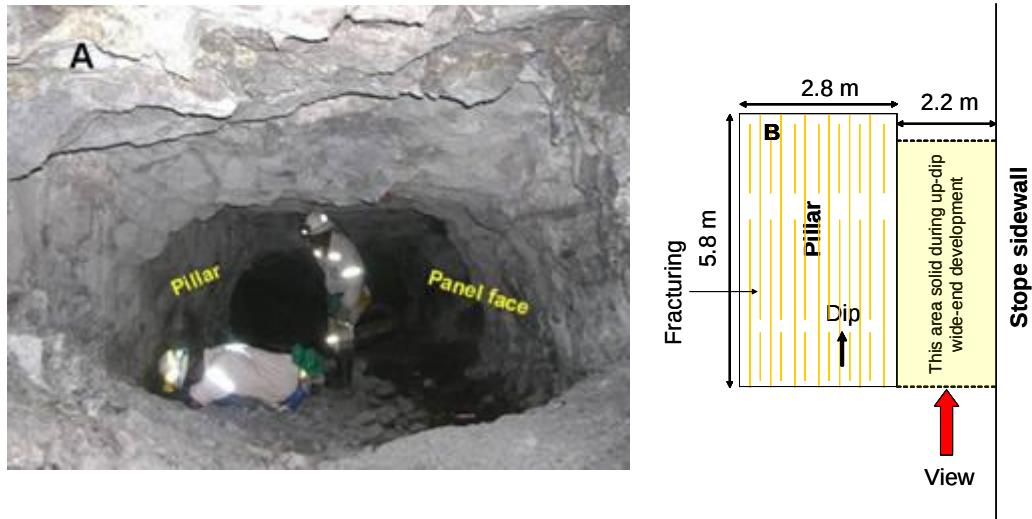


Figure 5-101 Union site: south side of P1 after pillar formation (A); sketch showing a plan view of the fracturing in the stub at failure and the view point of the photograph (B)

The severe fracturing observed on the north side of the pillar influenced the measurements at the P1a position (Figure 5-102). For this reason the results were not directly comparable to the P1b measurements, where the cell was located in a more favourable height and position above the centre of the proposed pillar. Thus the P1a measurements were not used to determine the peak pillar strength. The reason for the negative slope of the P1a measurements prior to pillar failure is probably because the peak stress in the pillar shifted away from the pillar edge with the progression of pillar fracturing. Since P1a was only 3.4 m above and 0.5 m back from the edge of the pillar (Figure 5-94), it was probably very sensitive to pillar fracturing as the peak stress migrated towards the pillar centre. P1b may also have been affected by this fracturing at it was only 5.4 m above the pillar. However, its location above the pillar centre may have resulted in a slight stress exaggeration.

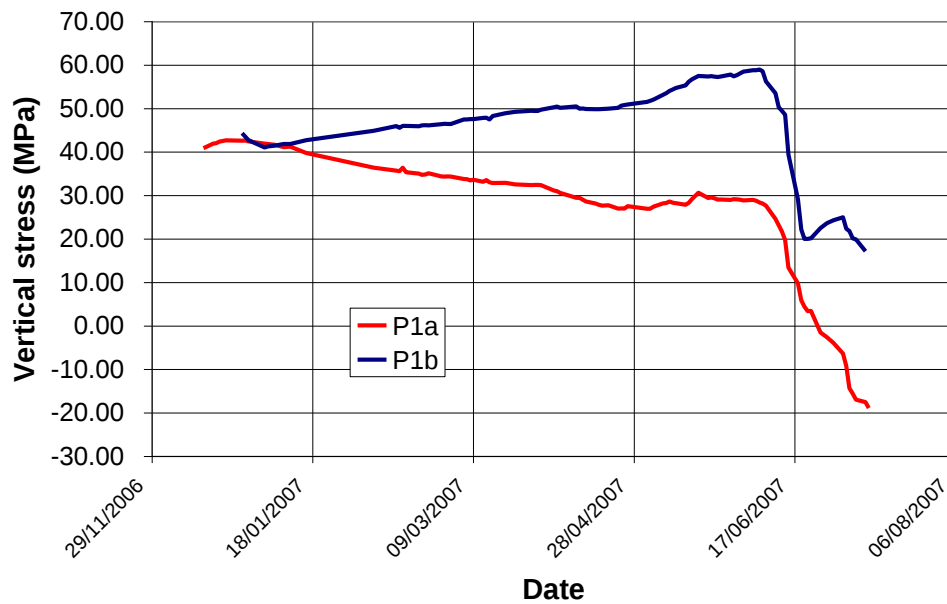


Figure 5-102 Union site: comparison between the vertical stresses measured at P1a and P1b

Assuming that P1b correctly reflects the APS (since it was located over the centre of the final pillar position) and that very little vertical stress is carried in the fracture zone; it was possible, with the aid of an elastic model, to track the failure progression or highest stress position in the pillar. This was done by finding the distance between the pillar edge and a point along a benchmark line in the model that represented the measured stress on the P1a straincell. The distance back from the P1a cell where the MinSim stress was equal to the measured stress was assumed to be the same as the depth of fracturing. Horizontal boreholes were drilled through the centre of the pillar for an extensometer and inspection purposes as shown in Figure 5-93. At the time of drilling the boreholes, fracturing was not visible on the sidewalls. However, some minor fractures were observed at a depth of about 0.42 m in both holes (Figure 5-103). Again, in both boreholes, borehole breakout extended along the boreholes from 0.42 m to 1.6 m. This suggests that the stress condition was relatively high between 0.42 m and 1.6 m, which is expected if this zone is relatively unfractured. The lack of borehole breakout between the pillar edge and a depth of 0.42 m suggests that this region was fractured and unable to sustain significant load. The analysis suggests that fracturing extended from 0.42 m at an almost linear rate of depth increase as the stress increased (Figure 5-104). At the time of pillar failure, the analysis

suggested that the fracturing had probably progressed further than the centre of the proposed pillar (Figure 5-105).

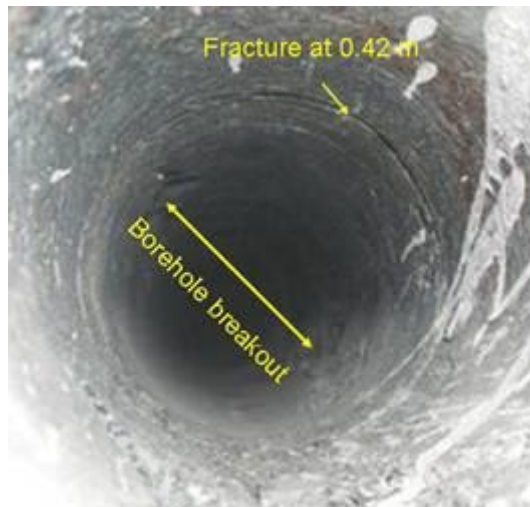


Figure 5-103 Union site: fracturing and borehole breakout observed in the horizontal P1 inspection borehole, drilled adjacent to the extensometer (Figure 5-100)

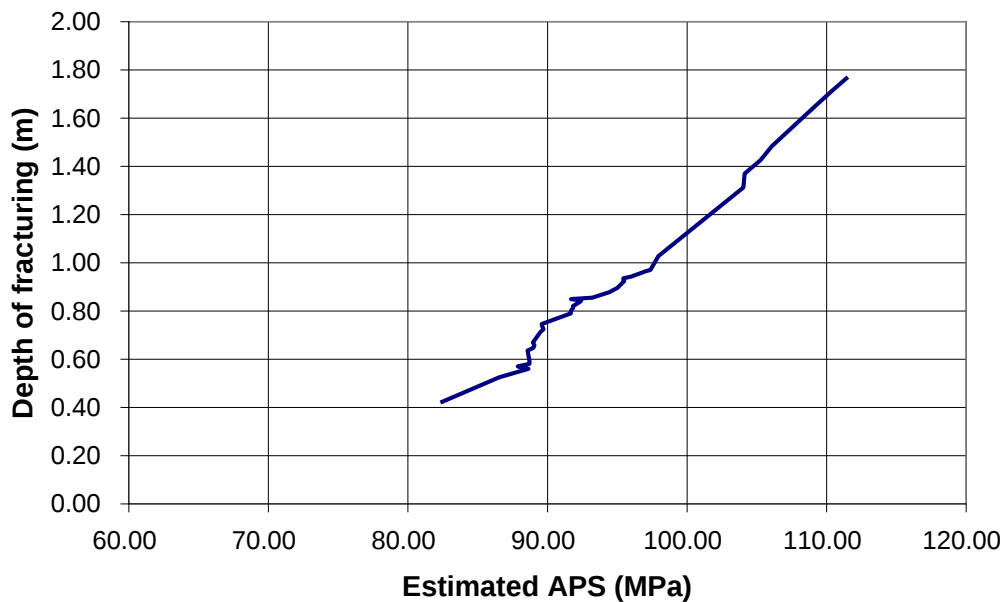


Figure 5-104 Union site: estimation of the depth of fracturing, based on a comparison between an elastic model and the measured stresses

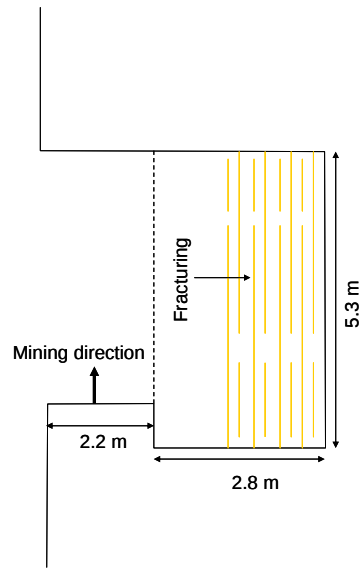


Figure 5-105 Union site: plan view of pillar showing the face position and fracturing just prior to failure (not drawn to scale)

The closure measured at 2.05 m from the pillar edge had a smaller component of inelastic deformation than the closure adjacent to the pillar (Section 4.4.3). Thus these measurements were used to estimate the pillar strain in Figure 5-106. The figure shows the estimated stress-strain behaviour of the instrumented pillar.

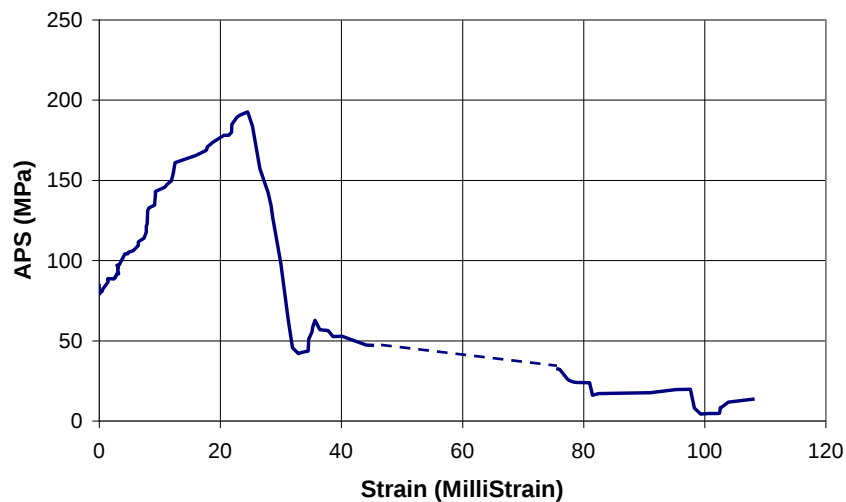


Figure 5-106 Union site: stress-strain curve for P1. Strain based on closure measurements at 2.05 m from the pillar edge

The sidewall of the proposed pillar was whitewashed to determine the onset and progression of fracturing (Figure 5-107). Minor fracturing, shown by the fragments

on the footwall (31 March 2007), was observed before mining commenced in Panel 3s. Fracturing of the corner was clearly visible on 9 April 2007. The fracturing extended up to the extensometer the following day. A sudden increase in pillar dilation rate was measured from 11 April 2007 (Figure 5-108). This change in dilation rate was closely followed by an increase in the stress-change rate (Figure 5-98) at about 104 MPa. The sudden increase in inelastic deformation shown by the extensometer (Figure 5-108) did not coincide with a change in mining conditions or the location of mining, but it appears that the APS condition (104 MPa) was high enough to cause significant sidewall fracturing. The closure-ride results (Figure 5-109) showed a change in the rates of footwall ride away from the pillar (relative to the hangingwall) from 7 May 2007. This coincided with a change in the closure rate and the initiation of mining on the down-dip side of the proposed pillar. It occurred at an APS of 145 MPa (Figure 5-98). The closure and ride results suggest the initiation of fractures or increase in fracture growth rate below the pillar at an APS of 145 MPa, i.e. foundation fracturing. Observations in the inspection borehole (Figure 5-110) drilled through the pillar prior to mining confirmed that the centre of the pillar moved downwards relative to the collar. The peak stress on 7 June 2007 and subsequent loss of load (Figure 5-98) coincided with a change in pillar dilation (Figure 5-108), closure and ride (Figure 5-109) as expected, confirming the date of pillar failure. Views of the sidewall condition just prior to pillar failure and subsequent to it are shown on 4 June 2007 and 31 July 2007 in Figure 5-107.



Figure 5-107 Union P1 fracture progression around the extensometer

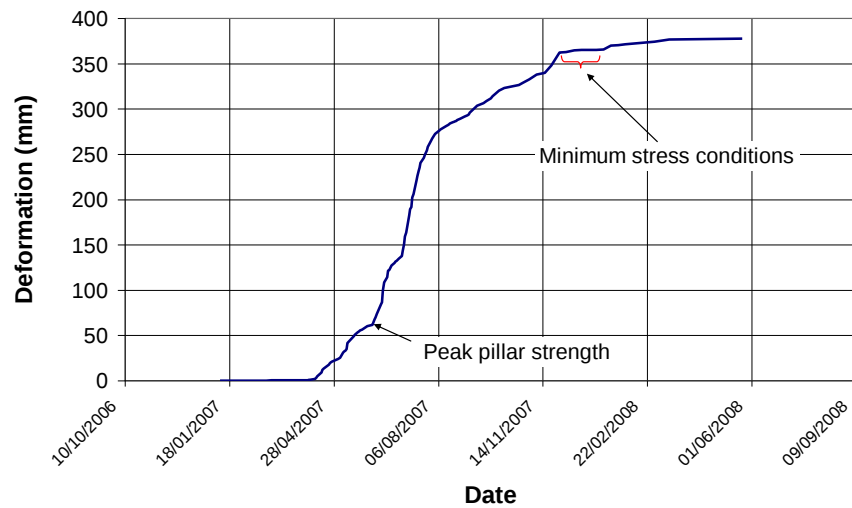


Figure 5-108 Pillar dilation, measured by a single-anchor horizontal extensometer through the centre of the pillar

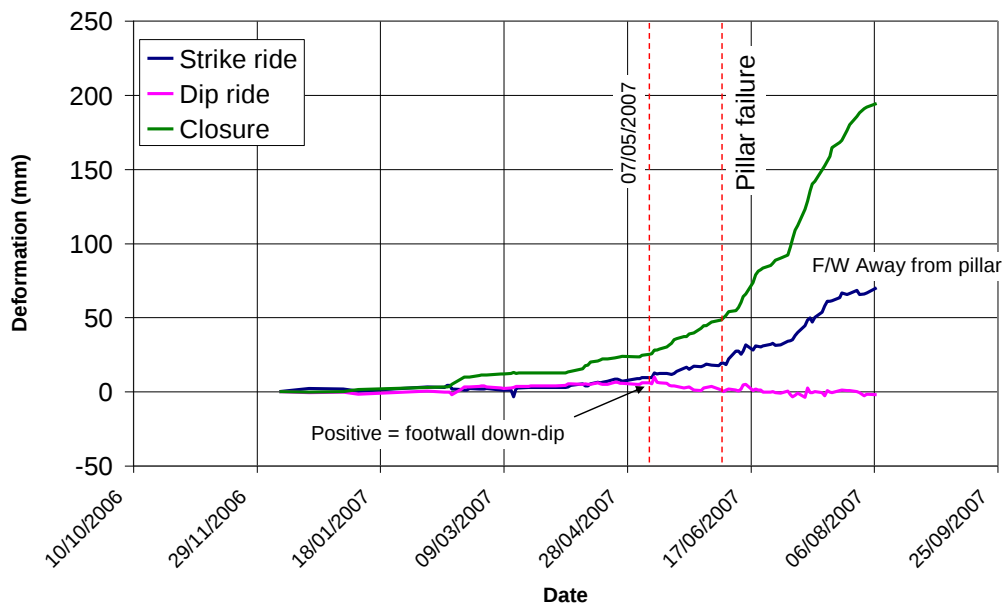


Figure 5-109 Closure-ride measured 0.15 m from Union P1



Figure 5-110 Deterioration of the inspection borehole in Union P1 before mining initiated (left) and just before pillar failure (right)

The pillar extensometer (Figure 5-108) shows that the pillar dilation became constant when the APS was at a minimum. However, once stress regeneration commenced the dilation increased again, but at a reduced rate. The closure measurements indicate that there was a relatively high closure rate at the time when pillar residual stresses dropped almost to zero (Figure 5-111), and there was stress regeneration when the closure rate was very low. This correlation between closure and pillar residual stress suggests that the pillar stress may have been affected by locking up of fractures/discontinuities in the foundation.

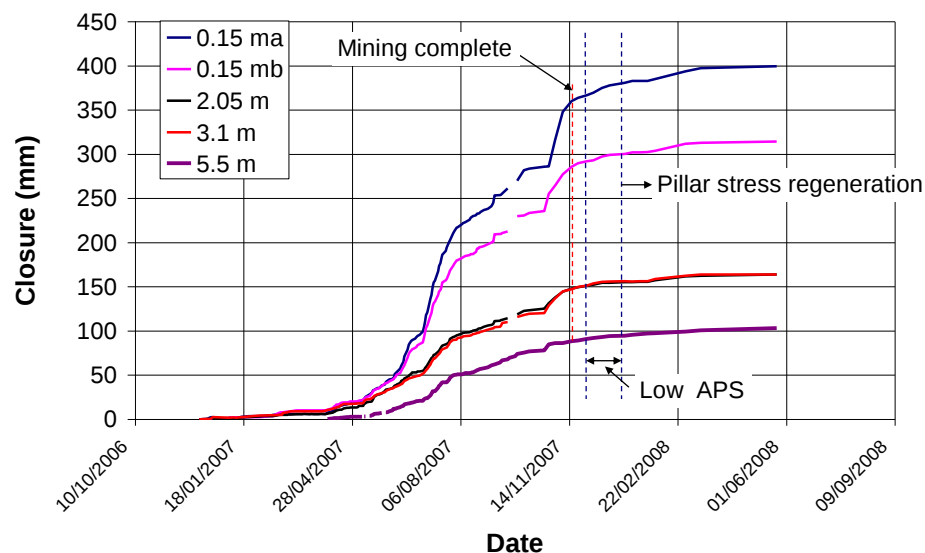


Figure 5-111 Closure measured in the ledge adjacent to Union P1. The time periods where the APS was only about 5 MPa and where the pillar stress was regenerated are highlighted

5.6.6.6 Union site: measurements of residual stress

After mining had been completed in Panel 3s, a series of stress measurements was conducted over the top of the pillar to determine the stress profile across the pillar. A shallow-dipping borehole was drilled from the south side of the pillar, parallel to the stope strike so that the measurements were at similar heights above the centre of the pillar – i.e. the horizontal measurements were made in the dip dimension (Figure 5-112). These measurements were required to confirm the residual pillar strength suggested by the stress-change instruments and to provide a stress profile across the pillar. The results of the investigation are shown in Table 5-19.

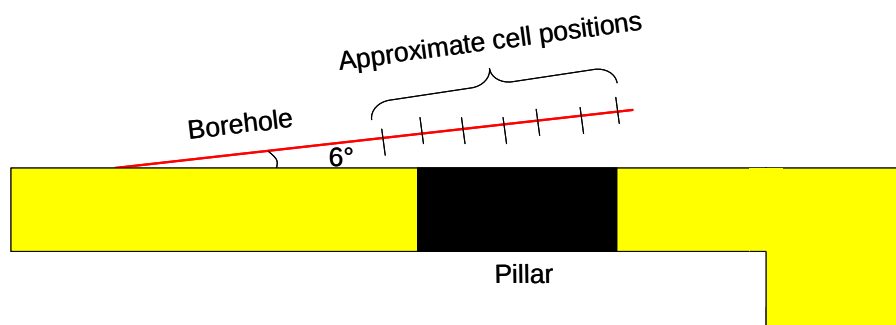


Figure 5-112 Union site: section showing the residual stress-profile measurement positions over P1 (not drawn to scale)

Table 5-19 Union site: stress-profile measurements across P1

Distance from pillar edge (m)	Height above pillar (m)	Error in strain measure. (%)	Vertical stress (MPa)
-0.91	1.45	5.8	2.7
0.09	1.56	1.6	19.0
0.53	1.61	8.7	43.8
0.97	1.65	0.0	6.2
1.48	1.71	3.6	6.4
2.02	1.76	18.4	5.5
3.00	1.86	9.5	15.8

Again, the measured stress distribution across the pillar could not be directly simulated using an inverse matrix of Boussinesq equations. Thus the pillar stress-profile evaluation was done using a 'best-fit' curve, as described in Section 5.6.3. The measurements, the 'best-fit' curve, the effects of a uniform pillar stress and the inferred stress distribution across the pillar from the 'best-fit' curve are shown in Figure 5-113. The measurements show that the highest stress was located above the pillar edge and very low stresses were located over the centre of the pillar. Under normal conditions of pillar formation, this is an unlikely scenario for a crushed pillar. However, since the formation and failure of this pillar was unusual, with its long axis perpendicular to the face advance, the pillar fracturing may also have been abnormal as suggested in Section 5.6.6.5. It is therefore possible that the pillar could be comparatively less fractured towards the south of the pillar in Figure 5-112 and Figure 5-113. The measurements were also affected by 'joint-channelling'. These two factors probably led to the uneven distribution of measured stress. Thus the evaluation of pillar stress shown in Figure 5-113 should be considered a rough estimate and the peak stress was probably slightly off-centre. The off-centre peak suggests that the pillar was slightly stronger on the side protected by the abutment at failure. This side of the pillar was observed to be less fractured than the other side of the pillar, suggesting that fracture density may have influenced the peak position. By inference, the height of the peak as well as the residual strength on this side of the pillar may have been affected by the fracture density.

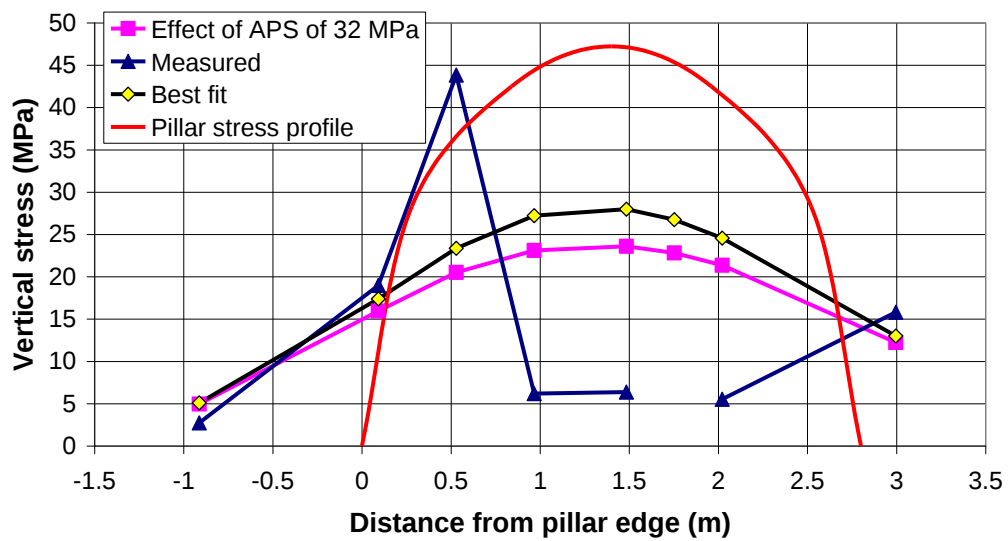


Figure 5-113 Union site: comparison between the measured, uniform stress (APS = 32 MPa) and ‘best-fit’ profiles across P1. The back-analysed pillar stress profile is also shown

The residual stress level shown by the stress-change cell at the time of the profile measurements was about 32 MPa (Figure 5-98). A back-analysis of a uniform stress of 32 MPa across the pillar (using the methodology described in Section 5.6.3) provided a profile of stresses at the locations of the measurement points, which were almost an average of the measured high and low stresses. This result suggested that the ‘best-fit’ stress profile across the pillar was probably almost uniform. The final ‘best-fit’ curve was a slight improvement on the uniform stress but also provided an average pillar residual strength of 32 MPa. The profile of stress across the pillar is shown in 3D in Figure 5-114. A comparison between the Wagner (1980) profile (Figure 5-7) and Figure 5-114 suggests that pillar failure was not complete and that the APS was likely to drop further – i.e. in the stress towards the edges of the pillar would probably drop. Figure 5-98 shows that this did occur.

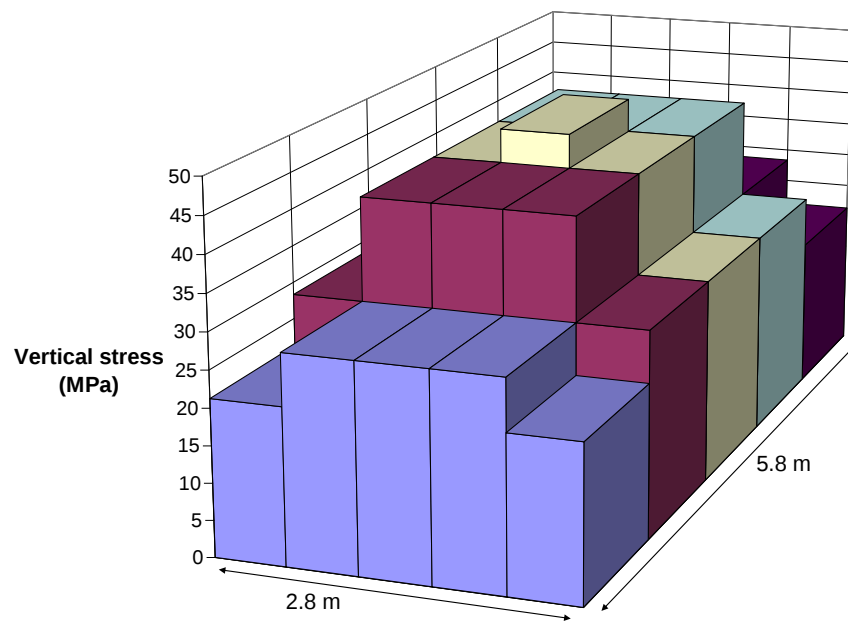


Figure 5-114 Union site: 3D stress profile across P1 (derived from the Boussinesq inverse matrix)

Field stress measurements were also made at 4.7 m and 5.4 m above the pillar. The results are shown together with Boussinesq back-analysis in Table 5-20. A reasonable correlation could only be achieved if it is assumed that the high measured stress was channelled along a 3.9 m long sub-vertical joint.

Table 5-20 Union site: comparison between the measured and back-analysed stresses in P1c

Height above pillar (m)	Measured (MPa)	Error in strain measure. (%)	Back-analysed assuming 'joint-channelling' (MPa)
4.66	28.9	8.4	28.6
5.38	11.7	9.3	12.8

5.6.7 Discussion

The previous sections of this chapter have dealt with a detailed description of the monitoring and interpretation of pillar behaviour. In the sections below, the information will be consolidated, leading to a pillar residual strength analysis and recommendations on the design of crush pillars.

5.6.7.1 Pillar behaviour

The peak pillar strength measurements are compared to the linear pillar strength formula (Section 5.3) in Figure 5-115. A good fit was shown by all the monitored pillars except the two highlighted pillars from the Amandelbult site, which are underestimated by the formula. Interestingly, the pillar-strength variation for a similar-size pillar was greater at the Amandelbult site and there also appears to be less foundation damage than at the other sites. The generally good correlation of the measured data with the formula strengthens the applicability of the linear equation to the calculation of pillar strength, particularly at the Impala and Union sites.

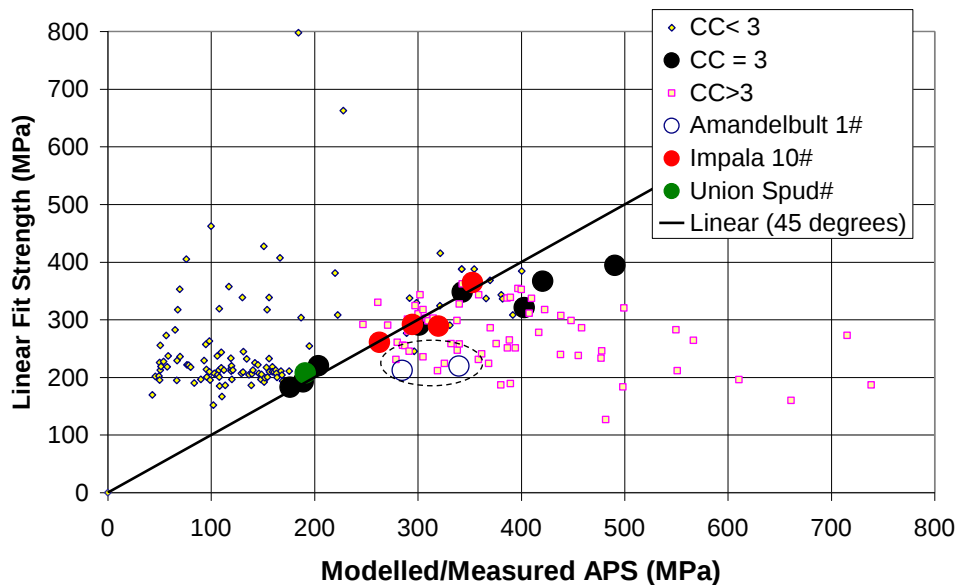


Figure 5-115 Comparison between the measured pillar strengths and the 'back-fit' pillar strengths, using the linear pillar-strength formula (Equation 5-9)

A comparison between the stress-strain relationships of all the measured pillars is shown in Figure 5-116. The peak strengths of all the pillars have been aligned in the figure by assuming the same strain at the peak strength to allow for a comparison of the post-failure behaviour. The initial post-failure behaviours were similar for all the pillars except Impala P2. The reasons for the relatively large initial strain in Impala P2 are discussed in Section 5.6.5.4. The good match between the other pillars suggests that the deformations measured between about 1 m and 2 m from the pillar edges were comparable. However, the extensometer through the centre of Impala P2 shows that the closure measured adjacent to the pillar was probably overstated due to bending of the strata and footwall heave. The similarity between the strain softening slopes of the Impala and Union pillars to the Amandelbult pillars (Figure 5-116), where the hanging- and footwall were almost unfractured, suggests that the inelastic strain contribution during the pillar unloading was probably small. Interestingly, there was a good correlation between the initial stress regeneration in Amandelbult P1 and Union P1 at about 100 millistrain.

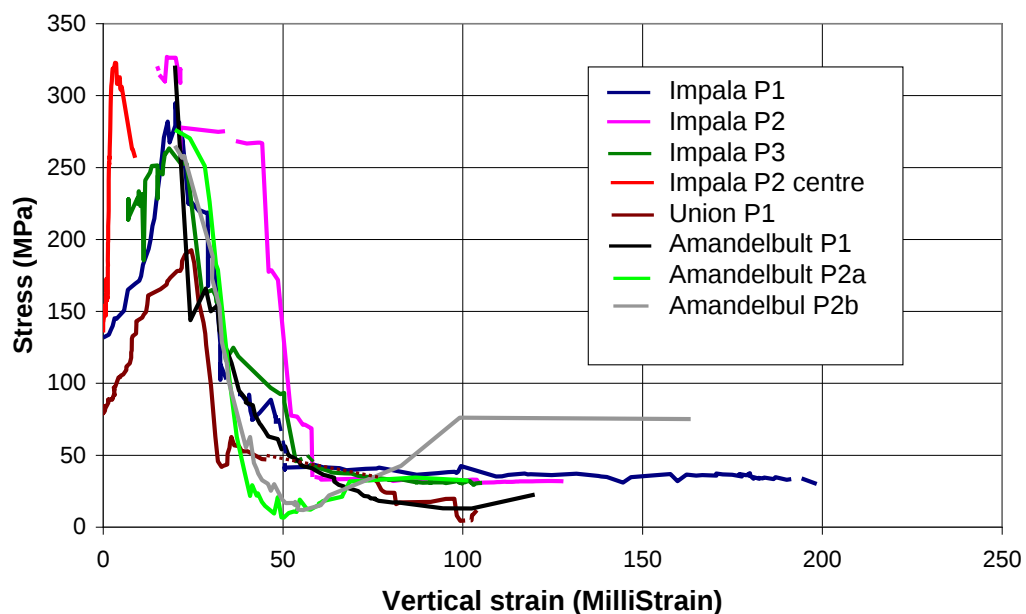


Figure 5-116 Stress-strain curves for all the instrumented pillars

The residual strengths of the measured pillars at the Impala site were all similar, even though the w/h ratios varied between 2.5 and 3.6. Once the residual state

was reached on pillars P1, P2 and P3, the stress remained constant for the duration of the monitoring period.

An additional pillar at the Impala site that is not shown in Figure 5-116 – Pillar S1 – seems to have had a slightly different unload curve to the other pillars after failure. This pillar was significantly larger than the other monitored pillars, but the residual strength appears to have been similar. Unfortunately, the straincell was installed well after failure, and no strain measurements were made adjacent to this pillar. The residual strength was determined by a triaxial field stress measurement conducted at approximately the same position as the stress-change cell.

The pillars at the Amandelbult and Union sites behaved differently from the 'crush' pillars at the Impala site. At the former two sites, there was an initial relatively rapid stress drop, followed by a slower drop to very much lower levels of stress than at the Impala site, and subsequently load regeneration. In most cases, however, the residual stress levels were higher than the minimum requirement of 1 MPa across the stope (Roberts et al, 2005), i.e. 13 MPa in a 30 m wide panel, while the workings were in operation. Only Amandelbult P2 appears to have dropped lower than the minimum residual strength, but subsequent stress regeneration meant that the average pillar stress in the pillar line was still greater than the requirement.

Possible explanations for the stress regeneration are:

- loading of the fractured pillar material by relatively less fractured foundations outside of the original dimensions of the pillar at failure, due to pillar dilation; and
- change in pillar w/h ratio due to pillar dilation and vertical shortening.

However, the same arguments apply to the Impala pillars, where stress regeneration was not observed.

Comparisons between the physical properties of the rock materials at the three sites were made to find an explanation for the different pillar behaviours (Figure 5-117). The pillars at both the Amandelbult and Impala sites were composite,

consisting of both hangingwall and footwall material. At the Union site, the pillars were similar to the hangingwall material. In all cases the hangingwall was slightly stiffer than the footwall; i.e. a higher Young's modulus, suggesting that more elastic deformations are likely in the footwall than the hangingwall. The only obvious difference between the Impala site and the other two sites is the comparatively higher strains at failure for both the hangingwall and footwall materials at the former site. This was the result of nonlinear behaviour and is unlikely to be the cause of the different pillar behaviours. Unfortunately, reliable post-failure laboratory tests were not available for the Amandelbult and Union sites, but previous tests performed at another Amandelbult site show that the anorthosite footwall is more brittle than the pyroxenite hangingwall.

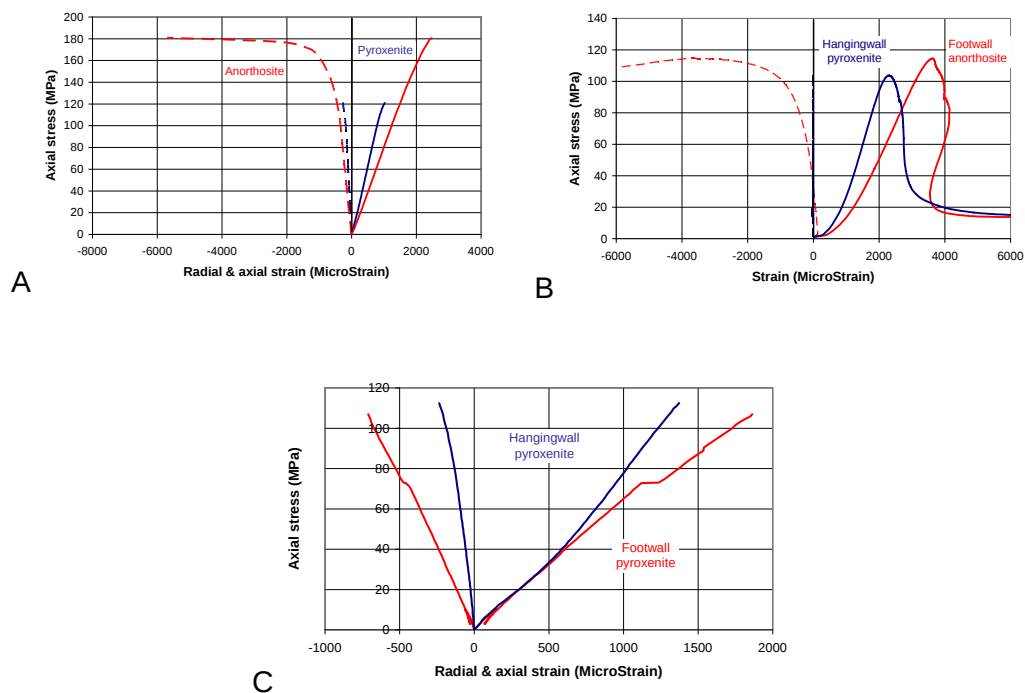


Figure 5-117 Comparison between the immediate hangingwalls and footwalls at: A Amandelbult site, B Impala site and C Union site

The behaviour of the pillars at Impala and Union sites may have been influenced by the fractured foundations. At both sites significant fracturing was observed in the stope footwall (Section 4.4), while the almost elastic closure measured at the Amandelbult site suggests minimal fracturing in the foundations at that site. The fractured footwall would have had a smaller restraining effect on pillar dilation than solid foundations would, effectively increasing the pillar height. The reduced

effective w/h ratio explains the relatively lower peak strength of the pillars at the Impala and Union sites as indicated in Figure 5-115. However, this mechanism does not explain the difference in the post-failure behaviour between the Impala site and the other two sites. It would seem that the effect of the fractured footwall on the residual pillar dilation is relatively greater than at peak strength because of the comparatively smaller pillar forces at residual strength. Thus, the measured w/h ratio of the pillars appears to be approximately equal to the effective w/h ratio at residual strength.

The only physical difference between the Impala site and the other two sites appears to be the presence of a shallow-dipping discontinuity that was observed in the footwall at the stope entrance at the Impala site (Figure 5-118). This structure may have influenced the fracture patterns in the footwall. Underground measurements of pillar behaviour performed by Spencer and York (1999) at Impala Platinum positively linked the influence of a strata-parallel discontinuity in the footwall to the depth of fracturing. However, FLAC modelling showed that a shallow-dipping discontinuity at this depth below the pillar would have little or no influence on pillar behaviour at a normal friction angle of 30°. Significant influence was only observed in the model when a friction angle of about 10° was assumed.



Figure 5-118 Strata-parallel fault observed in the travelling way, about 1.3 m below the stope

Another influence on the residual strengths at the Impala site may have been the loading conditions under which pillar failure took place. Impala P3 failed under relatively stiff loading conditions ahead of the face (Section 5.6.5), which may have influenced the fracturing and thus the residual strength. The other pillars at the Impala site were influenced by an ASG with variable depth. As the formula used for calculating the effects of the ASG on w/h ratio was based purely on

numerical modelling (Roberts et al, 2002), the higher effective w/h ratios for the Impala pillars may not be correct.

The closure during the very low pillar loads at the Amandelbult site was significantly more than that of the Impala pillars during the residual stress measurements. It could be argued that the closure adjacent to the pillar included some inelastic deformation that did not load the pillar. However, the extensometers adjacent to P2 at the Impala site showed that no inelastic deformation occurred either in the hanging- or footwall during this period. Similarly, the closure measurements at the Amandelbult site were shown to be almost elastic. A comparison between the closure at the Amandelbult and Impala sites, therefore, suggests that the loading rate is not the controlling factor for the lower residual strength at the former site. It appears that insufficient data was available to determine the true controlling influences on pillar post-peak behaviour and further research is therefore recommended.

The anorthositic footwalls at the Amandelbult and Impala sites are more brittle than the pyroxenite hangingwalls (Figure 5-117B and Section 4.4). In the presence of comparatively high horizontal stress, as suggested by the possible stress 'channelling' in the anorthosites, fracturing is more likely in the anorthosite footwall than in the pyroxenite hangingwall at these sites. The fractured footwall would thus be less resistant to pillar punching than the more solid hangingwall, even though the hangingwall at the Amandelbult site was significantly weaker than the footwall (Figure 5-117A). From the underground extensometer measurements and borehole camera surveys the footwall does appear to have been more damaged/fractured and weakened than the hangingwall at all three sites. However, during the initial stages of failure the Impala P2 pillar was shown to punch more into the hangingwall. Fractures were also observed above all the pillars where stress measurements were conducted relatively close to the top pillar contact. The reason for the initially greater punching of Impala P2 in the hangingwall and subsequent greater footwall damage shown by the instrumentation around Impala P2 is not clear. During the early stages of pillar failure the vertical borehole through the centre of Impala P2 sheared off. This effectively prevented further comparisons to be made between the hanging- and footwall behaviour. The general dominance of footwall punching by the pillars is

probably the result of a combination of the mining sequences, the influence of an ASG, spans, tectonic stresses, discontinuities and the material properties.

As the Impala pillars were composite, being cut with approximately 50% hangingwall pyroxenite and footwall anorthosite, and the Union pyroxenite pillar was of similar material to its foundation rocks, the assumption of the same material properties for the foundation and the pillars in the FLAC models is approximately correct for these sites. However, this assumption may be less applicable to the Amandelbult site, especially P1 that was cut into the hangingwall (i.e. a pyroxenite pillar on an anorthosite footwall). The strong footwall may be a reason for the slightly stronger pillars at the Amandelbult site.

5.6.7.2 Pillar residual-strength analysis

If it is assumed that a pillar consists of small vertical slices that are subjected to a uniform stress, it is possible to obtain a limit equilibrium solution for a perfectly plastic material that is yielding. According to Barron (1983), the vertical stress distribution through a pillar, consisting of such a material, can be expressed as:

$$S_{yy} = \frac{UCS_b}{2 \sin \phi_b} \left\{ (1 + \sin \phi_b) e^{\tan \phi_b \left(\frac{1 + \sin \phi_b}{1 - \sin \phi_b} \right) \frac{x}{h}} - (1 - \sin \phi_b) \right\} \quad 5-13$$

Where:

S_{yy} = Vertical stress within the pillar

ϕ_b = Internal friction angle of the broken rock

UCS_b = Uniaxial compressive strength of the crushed (broken) material

x = Horizontal distance into the pillar

h = Pillar half height

Figure 5-119 illustrates the pillar geometry, as well as the stresses acting on a vertical slice. The pillar width and height equal $2w$ and $2h$ respectively.

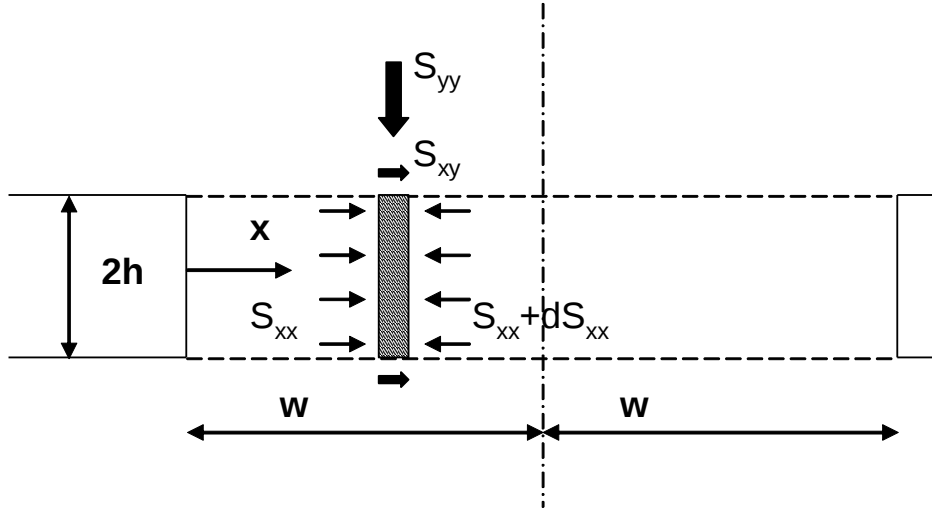


Figure 5-119 Geometry of the pillar section assumed in Equation 5-13

Equation 5-13 shows that the residual strength of a pillar with a given w/h ratio is dependent on the uniaxial strength (UCS_b) and internal friction angle (ϕ_b) of the broken rock. Assuming a constant ϕ_b , the pillar residual strength would be directly related to the UCS_b . The UCS_b is, in turn, dependent on the residual cohesion (C_b) and the ϕ_b (Mohr-Coulomb criterion):

$$UCS_b = \frac{2C_b \cos \phi_b}{1 - \sin \phi_b} \quad 5-14$$

Therefore, the residual cohesion is extremely relevant to the behaviour of a failed pillar. In addition, any horizontal stress applied at the pillar edge, e.g. support, has a similar effect. The residual cohesion can be thought of as a frictional effect due to gravity, which leads to small values in the kPa range. There may also be additional residual strength associated with interlocking of rough fracture surfaces. An equivalent residual cohesion of 0.011 MPa was estimated from the effects of gravity on a pile of rock at the edge of a pillar. Assuming $UCS_b = 0.038$ MPa, which relates to a cohesion of 0.011 MPa, and $\phi_b = 30^\circ$, the pillar stress (Barron, 1983) was plotted as a function of distance across the pillar and is shown in Figure 5-120. The results were also compared to the measured vertical stress profile of Amandelbult P1 in the figure. The analytical solution provided a much smaller profile than Amandelbult P1 and it was necessary to increase the UCS_b to 0.7 MPa to provide a comparable profile. Assuming $\phi_b = 30^\circ$, a cohesion of 0.2 MPa was calculated from Equation 5-14. The analytical solution suggests an exponential increase in vertical stress near the edge of the pillar. However, an

almost linear stress increase was generally interpolated from the measurements (Figure 5-120).

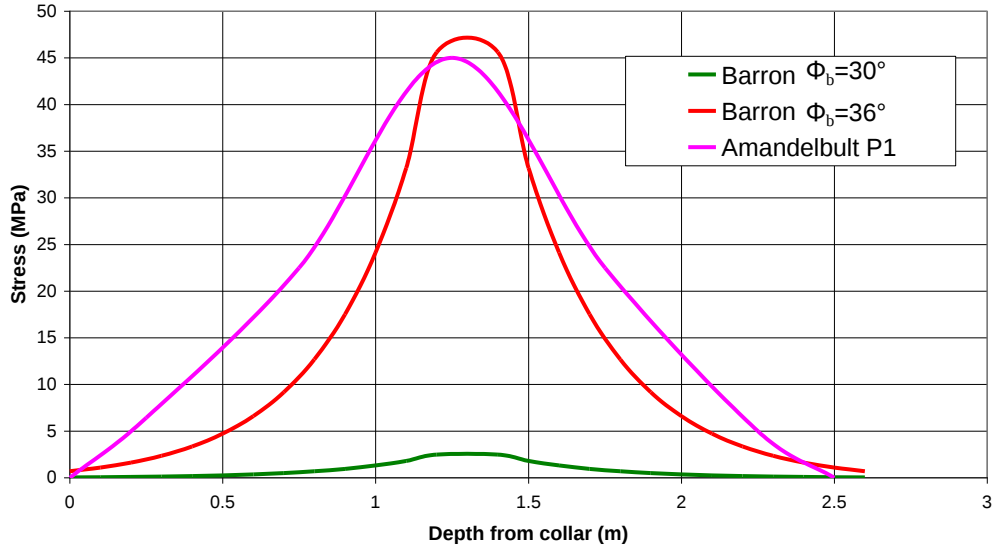


Figure 5-120 Comparison between the Barron formula (Barron, 1983) and the Amandelbult P1 stress profile. Assuming $\Phi_b = 30^\circ$ in the formula. 2.5 m-wide pillar in a stoping width of 1.1 m

Equation 5-13 was integrated by Kuipers et al (2008) to obtain the average residual pillar strength (APS_r):

$$APS_r = \frac{h}{w} \left[\frac{UCS_b}{2 \sin \phi_b} \left\{ \left(\frac{1 - \sin \phi_b}{\tan \phi_b} \right) e^{\tan \phi_b \left(\frac{1 + \sin \phi_b}{1 - \sin \phi_b} \right) \frac{x}{h}} - (1 - \sin \phi_b) x \right\} \right]_0^w \quad 5-15$$

Using the same parameters as determined for the curves in Figure 5-120, Equation 5-15 was plotted against w/h ratio and compared to the residual strengths measured underground and shown in Figure 5-121. The residual strengths measured in a laboratory by Spencer and York (1999) in their punch tests are also included in the figure. The solution that provided a reasonable fit to Amandelbult P1 in Figure 5-120 ($C_b = 0.2 \text{ MPa}$) does not fit the laboratory data and appears to overestimate residual strength at w/h ratios exceeding about 3. The solution for $C_b = 0.011 \text{ MPa}$ appears to underestimate the residual strengths even at comparatively high w/h ratios and does not compare with either the underground or laboratory results. However, both the Barron solutions (Barron,

1983) and the laboratory results show an exponential increase in residual strength with w/h ratio.

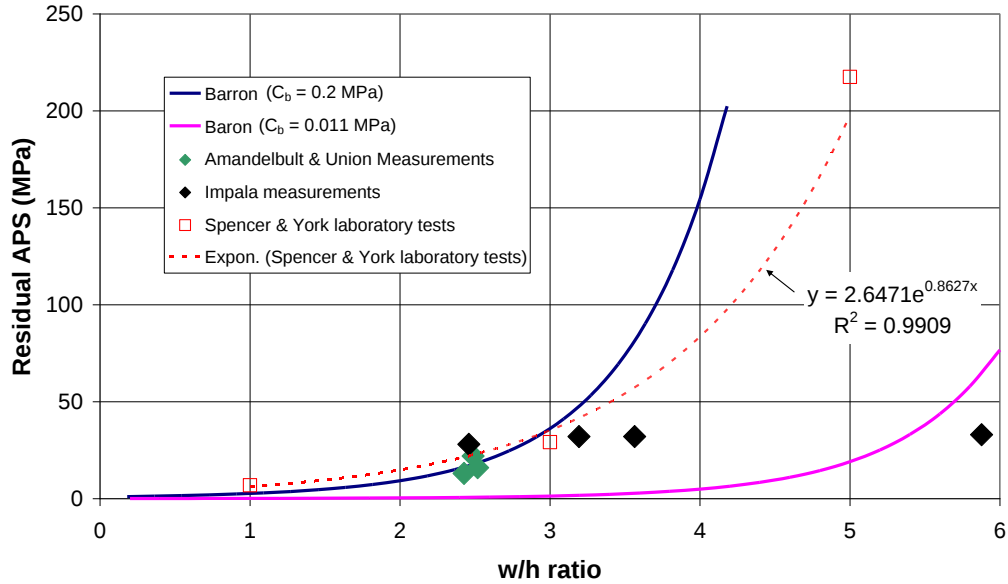


Figure 5-121 Pillar w/h ratio-strengthening effects on residual APS. The Barron (1983) solutions for $\Phi_b = 30^\circ$ are compared to the measurements and the Spencer and York (1999) laboratory tests

Salamon (1992) derived relatively complicated expressions to describe the stress distribution in a plastic pillar, that allow for a non-uniform stress distribution in the vertical slices, which is more realistic than the Barron solution (Barron, 1983) (Equation 5-16).

$$APS_p = \frac{2h \times 0.8 \frac{Co}{0.5}}{2.5w} \left[e^{2.5 \left(\frac{w}{2h} \right)} - 1 \right] \quad 5-16$$

Where:

Co = Cohesion

A friction angle of $\Phi_b = 30^\circ$ was suggested by the exponent in the relationship shown by the laboratory tests (Spencer and York, 1999) in Figure 5-121 and tilt tests performed on anorthosites. The curves for cohesions of 0.011 and 1.6 are plotted in Figure 5-122. For completeness the residual strengths measured in a laboratory by Spencer and York (1999), in their punch tests, and the results of relevant FLAC models are also included in the figure. These models were

conducted using the same strain-softening parameters as in the pillar-behaviour simulations, but assuming $C_o = 0.26$ MPa for the pillars and foundations.

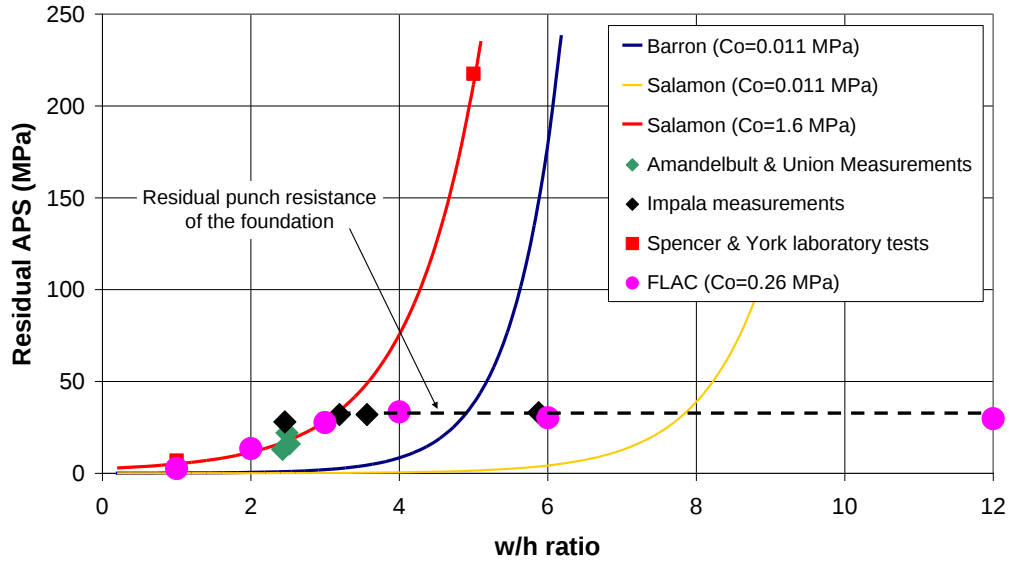


Figure 5-122 Pillar w/h ratio-strengthening effects on residual APS. The measurements are compared to the Barron (1983) and Salamon (1992) solutions ($\phi_b = 30^\circ$), Spencer and York laboratory tests (1999) and FLAC models ($\phi_b = 40^\circ$)

The curve resulting from Equation 5-16 fitted the underground measurements at the Amandelbult and Union sites as well as the laboratory tests (Spencer and York, 1999) when a residual cohesion of 1.6 MPa was assumed in the equation. Since the underground data is clustered within a very small range of w/h ratios, the suitability of the equation for describing the pillar residual strength at these sites may be questionable. The Salamon equation for stress distribution across the centre of a pillar (h) was applied, assuming $\phi_b = 30^\circ$ and a cohesion of 1.6 in Equation 5-17.

$$S_{yy} = 3.2 \left(0.8 \times e^{\left(\frac{2.5x}{2h} \right)} \right) \quad 5-17$$

The analytical solution was compared to the stress profile determined for Amandelbult P1 in Figure 5-123. It should be noted that neither the Barron (1983) nor the Salamon equation (1992) considers the effects of the foundations on pillar

behaviour; i.e. the foundations are considered to be elastic and the interface friction between the pillar and the loading platen is assumed to be equal to the internal friction of the material. Thus the equations predict “squat” pillar behaviour, which was not observed underground, as discussed in Section 5.4. In the analytical solution, a high w/h ratio would be associated with vertical stresses that are beyond the punching resistance of the foundation (hangingwall or footwall). This is an explanation why the larger w/h ratios may not be well represented by the analytical solutions shown in Figure 5-122.

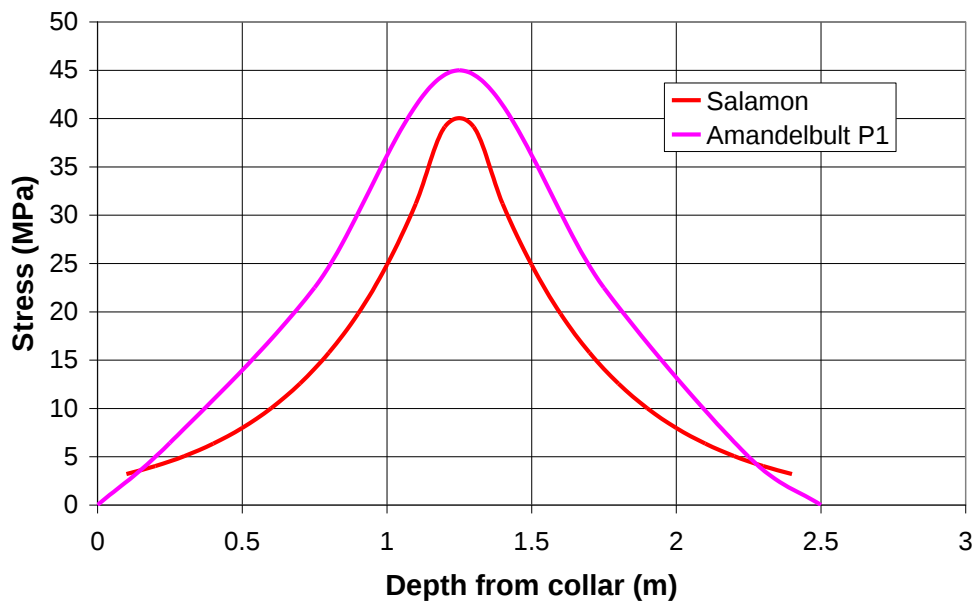


Figure 5-123 Comparison between the Salamon formula (Equation 5-17), assuming $\Phi_b = 30^\circ$ and $Co = 1.6$, and the measured Amandelbult P1 stress profile

The laboratory tests conducted by Spencer and York (1999) were carried out using a cylindrical punch of 25 mm diameter and a foundation cylinder of 80 mm for the diameter and the length. The foundation was confined by a metal ring. Both the punch and the foundation were anorthositic norite from the immediate Merensky Reef footwall at Impala Platinum. This material is typical of the lower half of the pillars and immediate footwall of the Merensky Reef at Impala Platinum.

The w/h ratio of the punch was varied by changing the punch-height. The boundary condition at the top of the punch controls the effective w/h ratio. If the interface between the punch and the metal platen were frictionless, the w/h ratio would be halved as this interface acts as an axis of symmetry (absence of shear stress). Since this interface had a friction angle of about 12° (York et al, 1998), it could neither be regarded as a plane of symmetry nor a rough interface similar to the one between the punch and the foundation. The true w/h ratio will therefore be greater than 50% of the actual w/h ratio of the punch, but the effective ratio cannot be quantified with any certainty. This issue should be considered in the analysis of the results. The peak strengths of the laboratory tests (Figure 5-124) were similar to the strengths provided by the FLAC results, which were discussed in Section 5.4. The linear peak strength formula (Section 5.5) also indicated similar pillar strengths for the corresponding w/h ratios.

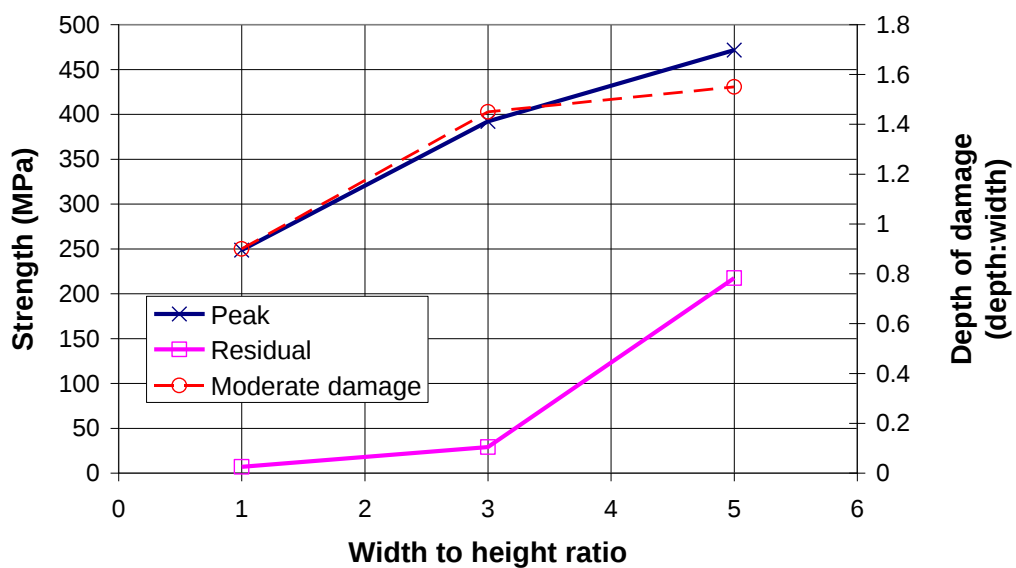


Figure 5-124 Results of laboratory punch tests (after Spencer and York, 1999)

The initially higher rate of increase of peak strength with w/h ratio in Figure 5-124 was also shown by the FLAC modelling in Section 5.4. A drop in the rate of strength increase at higher w/h ratios indicates pillar punching, assuming the punches, foundations and confinements provided to the foundations by the metal rings in the three tests were all the same. This punching was also illustrated by

the comparatively large increase in the depth of damage observed between w/h ratios of one and three in the laboratory tests (Figure 5-124). The comparatively smaller increase in the depth of damage between w/h ratios of three and five may be due to the mobilisation of Prandtl wedge-type fractures (Prandtl, 1921) at the higher w/h ratios.

It should be emphasised that there are differences between laboratory tests and analytical solutions. For instance, the analytical solution is based on the limit equilibrium of the material that is loaded between two solid platens; in contrast, the punch test allows for damage in the foundation. This foundation damage can have serious implications for peak pillar strength as has been demonstrated in Section 5.4. It is very plausible that residual strength is also affected by foundation damage. In addition, the formulae are based on plane strain conditions, whereas the laboratory tests were axisymmetrical. This geometric effect will also result in different relationships between w/h ratio and strength.

Despite the differences, an attempt was made to match the laboratory results with the Salamon analytical model (Equation 5-16). Since the Salamon equation (Equation 5-16) overestimates the stress around the core of the pillar (Figure 5-123), the average pillar stresses may also be overestimated. This would be particularly true for larger w/h ratios. In order to match the Salamon equation with the results of the laboratory tests, a relatively low effective friction angle has therefore to be selected in this equation. The actual material friction angle can thus be expected to be higher.

It can also be argued that the w/h ratio of the laboratory specimens was affected by the boundary conditions, especially the interface between the steel platen and the small rock disc. The limited friction along this interface would cause a decrease in the effective w/h ratio of the disc because of a reduced clamping effect. If it is assumed that this decrease could be as much as 25%, the parameters for the Salamon equation (Equation 5-16) need to be adjusted. A sensitivity analysis showed that the friction angle and the residual cohesion could vary between 25° and 31° and 0.7 MPa and 1.6 MPa, respectively, to match the laboratory results with the equation.

The magnitude of the calibrated internal friction angle seems realistic, especially since the actual material friction angle can be expected to be a bit higher as explained above. The cohesion is much higher than would be expected of a crushed material. One possible explanation is that the material is not completely crushed and that the broken rock actually maintains a surprisingly high residual strength. The pillar centre may also be comparatively more ductile and therefore less fractured as a result of confining stresses that could develop here. The effect of confinement on brittleness was researched by Fang and Harrison (2002). The residual strength resulting from friction between blocks, held together by gravity, would only account for about 1% of the calibrated value. The relatively high strength of broken rock has not been reported previously but it is an extremely important parameter, especially for the design and behaviour of 'crush' pillars. Factors influencing the residual strength may be: interlocking blocks, block size, failure violence, peak strength and residual cohesion. These factors may not be constant. However, a surprisingly consistent stress drop to residual strength at about 50 millistrains was noted for a range of pillar w/h ratios and loading scenarios.

The uncertainty of the relatively high residual cohesion-mechanism is of concern as adequate residual strength may not always be present. This inadequate residual strength was also suggested by the stress drop that occurred after mining had been completed at the Union site (Figure 5-98).

An attempt was made to simulate the behaviour of a perfectly plastic pillar with FLAC. The model consisted of crushed material loaded between two rough platens. Unfortunately, no unique solution could be derived as the mesh density affected the results to such an extent that an infinitely dense mesh would result in a zero resistance. Note: that this applies to the non-associative flow rule ($\Phi > \Psi$) only, for which the analytical solution does not apply. To the author's knowledge, this issue has never been reported before and needs to be resolved.

Neither the analytical nor the laboratory results matched the linear relationship between pillar residual strength and w/h ratio as measured underground at the Impala site (Figure 5-122). Since the pillars at this site represent mainly the

higher w/h ratios, the effects of the fractured foundation bearing capacity were investigated using an analytical solution for bearing capacity and FLAC.

The ultimate bearing capacity of the pillar foundation is dependent on the cohesion and friction and dilation angles of the foundation material. No further increase in residual pillar strength can be expected above the bearing capacity. The results in Figure 5-122 suggest that the effective range of w/h ratios is limited to a maximum of about three. No further increase in strength can be expected for greater w/h ratios. The relationship between cohesion (C_0) and bearing capacity (BC) for a given friction angle is shown in Equation 5-18 (Meyerhof, 1951). The dilation and friction angles are assumed to be the same in the equation (associative flow rule).

$$BC = \left[\left[e^{\pi \tan \phi} \tan^2 \left(45 + \frac{\phi}{2} \right) \right] - 1 \right] \cot \phi \times C_0 \quad 5-18$$

Bearing capacity has been plotted as a function of cohesion for friction angles of 30° and 40° in Figure 5-125. The ultimate bearing capacity of the pillars as shown in Figure 5-122 appears to be about 33 MPa. This suggests cohesions of about 1.1 MPa or 0.4 MPa for friction angles of 30° or 40°, respectively. The FLAC models indicated a cohesion of 0.3 MPa at a friction angle of 40°. The difference between the model and the analytical solution is probably because the associative flow rule was not assumed in the model. The dilation angle in the model was 10°.

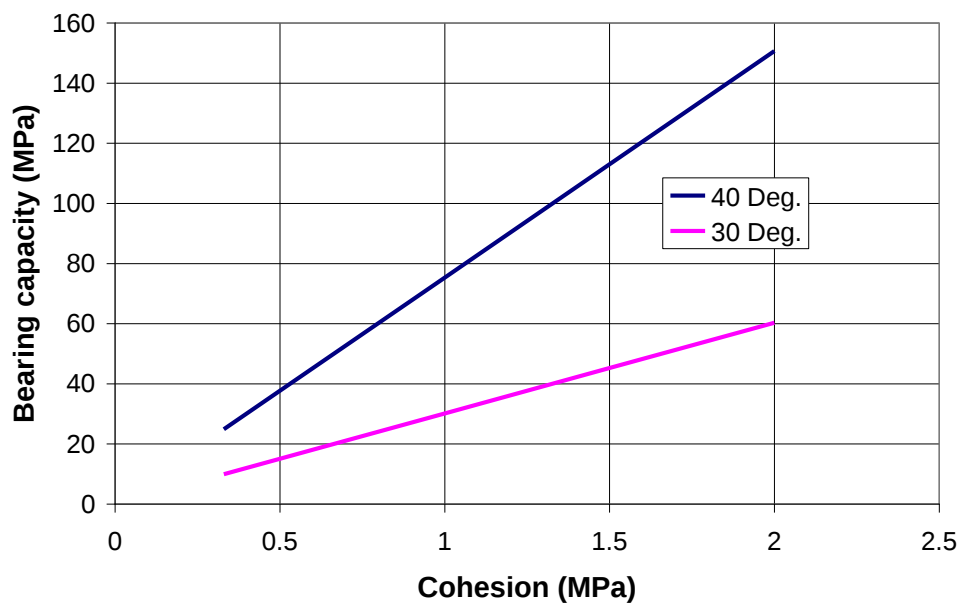


Figure 5-125 Ultimate bearing capacity of the foundation, assuming friction angles of 30° and 40°

The good agreement between Equation 5-16 and the residual strength from the laboratory results as given in Figure 5-122 suggests that pillar punching either did not occur or was restricted in the laboratory tests. This was probably true for the two smaller w/h ratios as there is reasonable correlation between the laboratory, underground and FLAC results. However, the residual strength of the rock punch with a w/h ratio of 5 was significantly different from the other results even though there was a good correlation between the peak strength and the underground results. The peak results from the laboratory tests appear to suggest punching, while the residual laboratory test results suggest that punching is not occurring. This apparent contradiction can be explained in several ways:

- Relatively large post-failure punch dilations, in the high w/h ratio sample, resulted in stress being generated in the dilated fractured sidewall between the metal loading platen and a solid foundation;
- Interference of the boundary on the results (the foundation cylinder was only three times larger than the solid punch); and
- The grain size is large in comparison to the height of the punch. Post-failure behaviour may therefore have been influenced by fractures developing through relatively strong grains.

The initial FLAC models were run with an assumed zero residual cohesion. This approximation resulted in a reasonable correlation with the results of the peak pillar strength back-analysis shown in Section 5.5.9. The residual strength results were, however, different from the underground and laboratory results, and indicated a linear residual strength increase with w/h ratio. It was found that the residual strength was influenced by the loading velocity, which dropped to zero strength at zero loading velocity under conditions of zero residual cohesion (Figure 5-126). Once the original loading conditions were resumed, the residual strength returned to the same level. This indicates some form of “plastic” flow resistance associated with the loading velocity. Obviously, this factor needs to be excluded to obtain more realistic results.

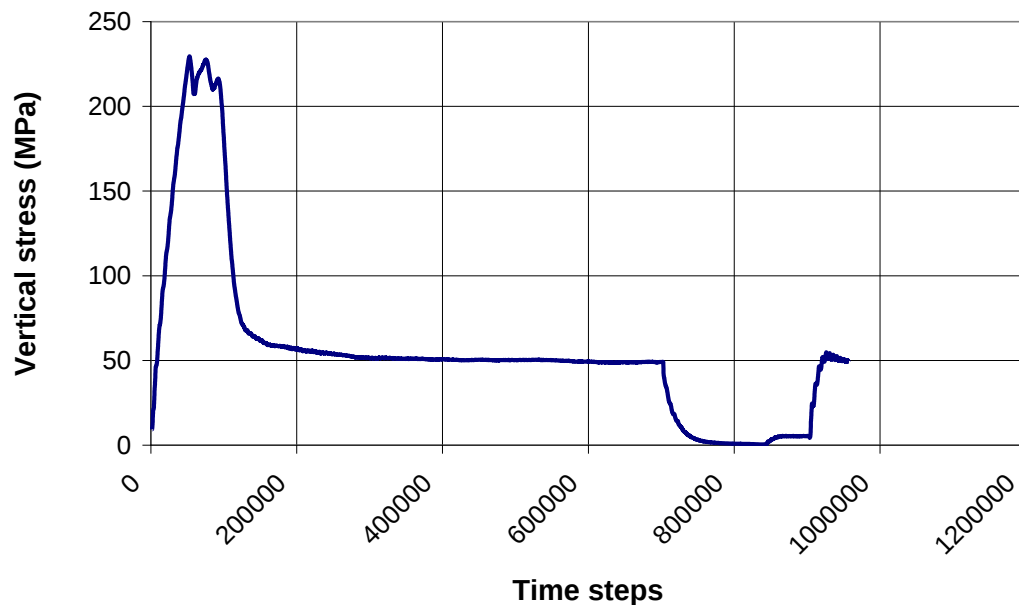


Figure 5-126 Stress-time-step curve showing the effect on residual strength when the loading velocity was dropped to zero

A second set of FLAC models was run using the original material parameters but assuming that the cohesion did not drop to zero at residual strength levels. A residual cohesion of 1 MPa was used together with a loading rate that was reduced to zero at the end of the runs. The results are shown in Figure 5-127. A significant drop in stress occurred when the loading rate was reduced to zero, but in these models a more realistic residual strength was obtained. The subsequent very slow loading rate also did not change the residual strength. These models

showed that there is an almost linear increase in residual strength with w/h ratio up to a w/h ratio of about 3 (Figure 5-122). Although the models appear to have reached the bearing capacity of the foundations at a w/h ratio of 3, the peak strength increased with w/h ratio to a w/h ratio of about 10 as discussed in Section 5.4. The peak strengths were again modelled, including the residual cohesion and a very slow loading rate. There was very little difference between the results of former and latter models on the peak pillar strength.

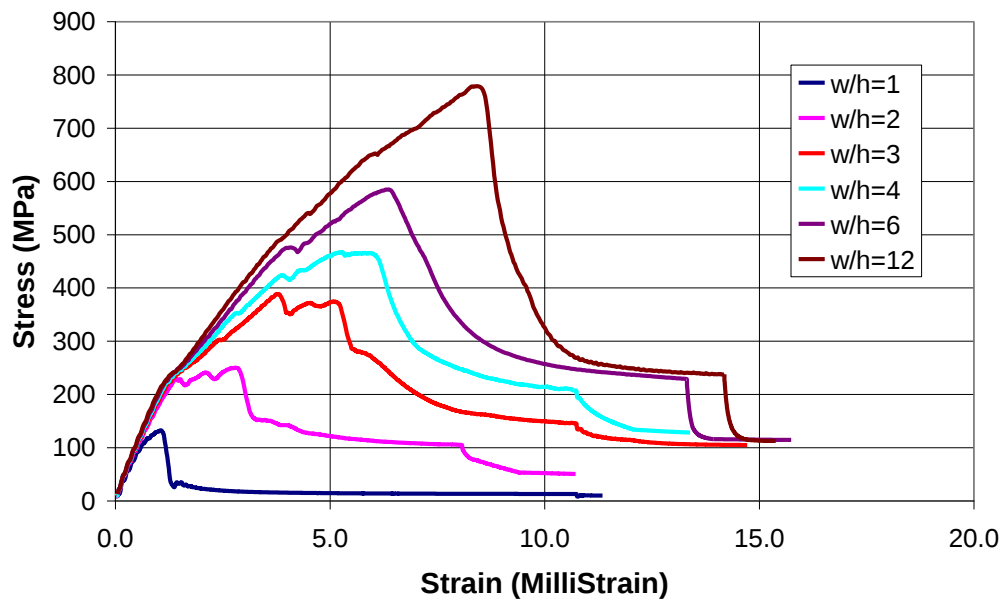


Figure 5-127 Stress-strain curves of pillars with different w/h ratios. Generated by FLAC, assuming a constant width and varying the height

A good match between the underground results, the two lower w/h ratio laboratory tests and the FLAC models was shown if a cohesion of 0.26 was assumed in the model. These FLAC results have been included in Figure 5-122.

Vertical fractures extended parallel to the vertical grid orientation. This is unfortunately a numerical problem, which did not seem to affect the results seriously. The final residual condition appeared to be dominated by a column of fractured material, about the width of the pillar and extending vertically into the foundation. Finally, the friction angle of 40°, which may be applicable to the pre-failed pillar, may be too high for the failed pillars.

There are several reasons why the residual strength of Impala P3 was greater than for the other similar-sized pillars at the Amandelbult and Union sites:

- The w/h ratio may have been misrepresented for Impala P3 due to the very small stoping height in some places along the siding on the up-dip side of the pillar;
- The fracture development in the pillar and foundations may have been influenced by the unusually stiff loading condition under which failure took place; and
- The fracture development may have been influenced by the shallow-dipping discontinuity in the footwall.

The arrangement of the fractures may have affected the residual cohesion by providing greater friction between the sliding fracture surfaces. Interestingly, though, the final Amandelbult P2a residual strength, after strength regeneration, was similar to that of the Impala pillars (Figure 5-116). Further investigations are required to determine the reasons for the generally high post-failure cohesion required to simulate the residual pillar strengths measured underground. This may include the development of a more appropriate post-failure strength criterion.

5.6.7.3 Design of ‘crush’ pillars

The main objective in ‘crush’ pillar design is to ensure that the residual strength of the ‘crush’ pillars is sufficient to arrest a backbreak. In the Bushveld platinum mines, the criterion is 1 MPa across the stope (Roberts et al, 2005). This is achieved if the residual strengths of pillars are between 8 MPa and 13 MPa and the pillar lines are spaced 30 m apart. Pillar size should therefore be designed with the residual strength in mind. However, several recent pillar bursts at Amandelbult, with serious consequences, show that the peak strength also needs to be considered in the design. Pillars that are too strong may not fail near the face where the loading conditions are stiff. Three of the pillar bursts at Amandelbult were investigated to provide some insights into acceptable loading stiffnesses (Watson et al, 2007b). All the pillar bursts occurred on the first or second pillar (>10 m) back from the lagging face as shown in Figure 5-128. One of these bursts created a magnitude 1.2 seismic event, resulting in violent ejection of rock as shown in Figure 5-129. This pillar was located 10 m to 14 m

behind the lagging face at the time of the burst. A high degree of fragmentation was observed throughout the pillar and rock fragments were thrown into the ASG.

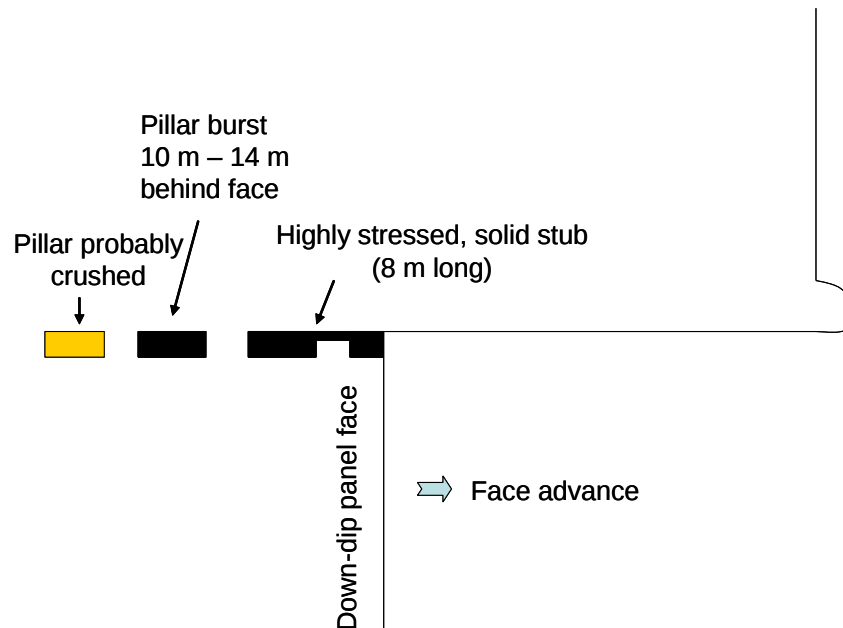


Figure 5-128 Plan view showing the face position where the investigated pillar bursts occurred (not drawn to scale)



Figure 5-129 Panoramic view showing the up-dip end of a burst pillar. Note the gully is full of rock fragments from the pillar burst

Most of the instrumented pillars failed in line with or just behind the lagging face. Only Amandelbult P1 and Impala P1 failed as stubs about 4 m to 5 m behind the face as shown in Figure 5-130. Pillar P0 at Amandelbult failed in a semi-stable manner at about 7 m behind the lagging face (Figure 5-131). As all the pillars failed stably with only a minor ejection of rock from Impala P1, loading conditions at 5 m to 7 m from the lagging face appear stiff enough. However, evidence from

the pillar bursts suggests that unfailed pillars located at 10 m or more from the face are in a dangerous, soft-loading situation and may burst if failure takes place.

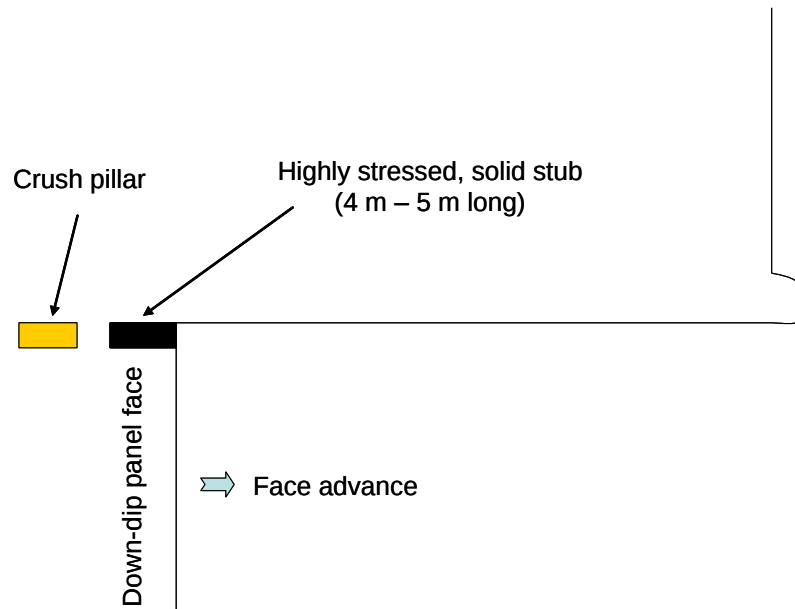


Figure 5-130 Plan view showing the face position where Amandelbult P1 and Impala P1 failed (not drawn to scale)

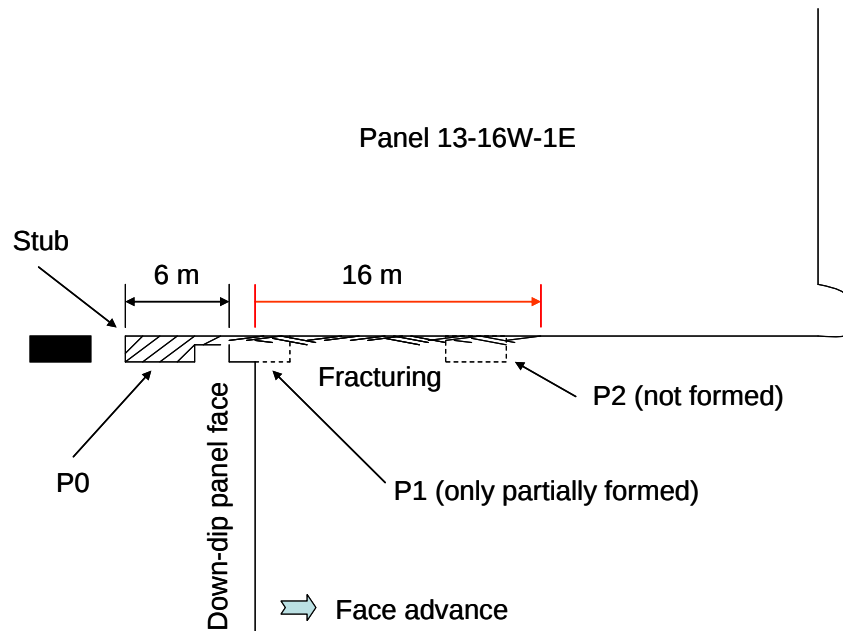


Figure 5-131 Sketch showing the fracturing that occurred when P0 failed

From the evidence of the few collapses referred to above, surrounding strata stiffness for acceptable and unacceptable loading conditions have been determined from elastic modelling (Figure 5-132). The elastic acceptable deformation in the figure is based on the average deformation across the area of the pillar at a face advance of 4 m (Figure 5-130). The adjacent pillars in the investigation were assumed to have a residual strength of 20 MPa. The unacceptable loading conditions refer to the same scenario as the acceptable model but for conditions where the pillar fails at 8 m to 10 m behind the face. The underground lines refer to measurements conducted about 2 m down-dip from the pillar edge and at about the mid point of the pillar. Most of the measured pillar load-deformation curves were almost parallel to the elastic acceptable line. None of the instrumented pillars reached a deformation of 32 mm during the initial failure and all failed in a reasonably stable manner. The unacceptable stiffness of the surrounding strata is about 5.0 mm/GN. Most of the measured pillars failed under loading conditions where the stiffness of the surrounding strata was about 3.2 mm/GN

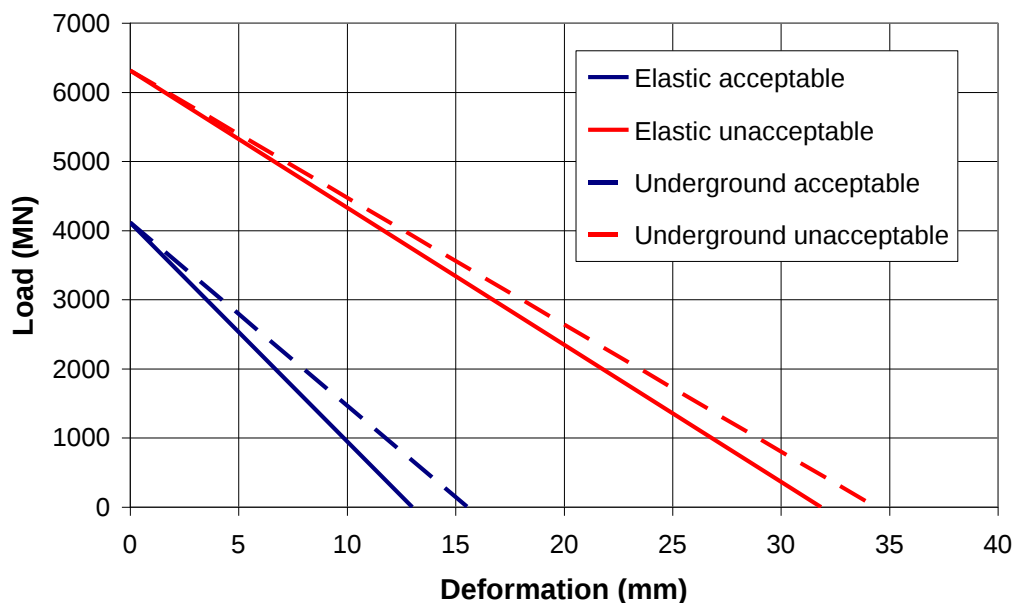


Figure 5-132 Acceptable and unacceptable stiffness of the surrounding strata for stable ‘crush’ pillar design. Elastic = average over pillar, underground = measured adjacent to pillar

The pillar strength back-analysis described in Section 5.3 suggests a direct relationship between peak pillar strength and w/h ratio up to a w/h ratio of about 8. Larger pillars are thus less likely to fail near the face than smaller pillars. Pillar strength should not exceed the available loading capacity of the system within 5 m of the face. At shallow depths this would require very slender pillars that may not have sufficient residual strength to stop a backbreak. Under these conditions pillar preconditioning could be an option. A flowchart for the design of ‘crush’ pillars is provided in Figure 5-133. The process assumes that a suitable residual strength is calculated on the basis of strength requirements (1 MPa across the stope). The approximate pillar w/h ratio to support the residual strength requirements can be determined from Figure 5-134. The calculations should include panel spans between pillars rather than a pure extraction ratio. The peak strength is determined using Equation 5-9, which has been reproduced in Equation 5-19. The latter equation includes the substitution of the factors obtained by the back-analysis described in Section 5.5. As the formula provides the pillar strength at a safety factor of unity, the pillars have a 50% probability of failure. This probability can be increased by ensuring that the strata loading capacity is greater than suggested by the formula. The graphs in Figure 5-135 show a comparison between the safety factor and the additional loading capacity requirements for pillars with w/h ratios of 2.0, 3.0 and 4.0. The additional stress, corresponding to the desired probability of failure, should be added to the stress suggested by Equation 5-19 to determine the loading requirements.

$$PS = 136 \left[\frac{1.27}{1 + \frac{0.27w}{L}} \right] \left[0.59 + 0.41 \frac{w}{h_e} \right] \quad \mathbf{5-19}$$

An elastic model should be run to determine if the loading capacity is sufficient to fail the required pillar within 5 m of the face. Should the loading capacity be insufficient to fail the pillars, either preconditioning or redesign of panel spans should be considered. Smaller spans will require smaller residual strengths and thus smaller pillars. The elastic model and process should be repeated with the new layout. ‘Crush’ pillars located adjacent to stability pillars or potholes (left as stability pillars) may need to be cut smaller than other pillars in a panel to ensure correct failure conditions.

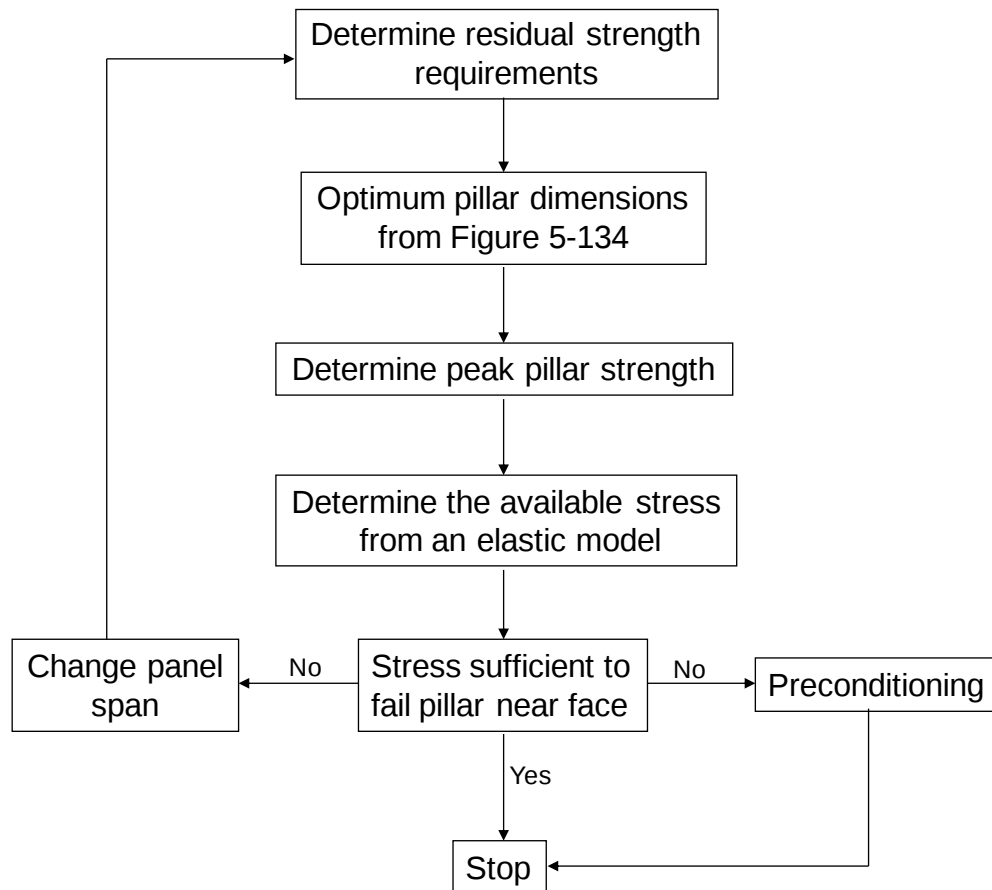


Figure 5-133 Flowchart for 'crush' pillar design

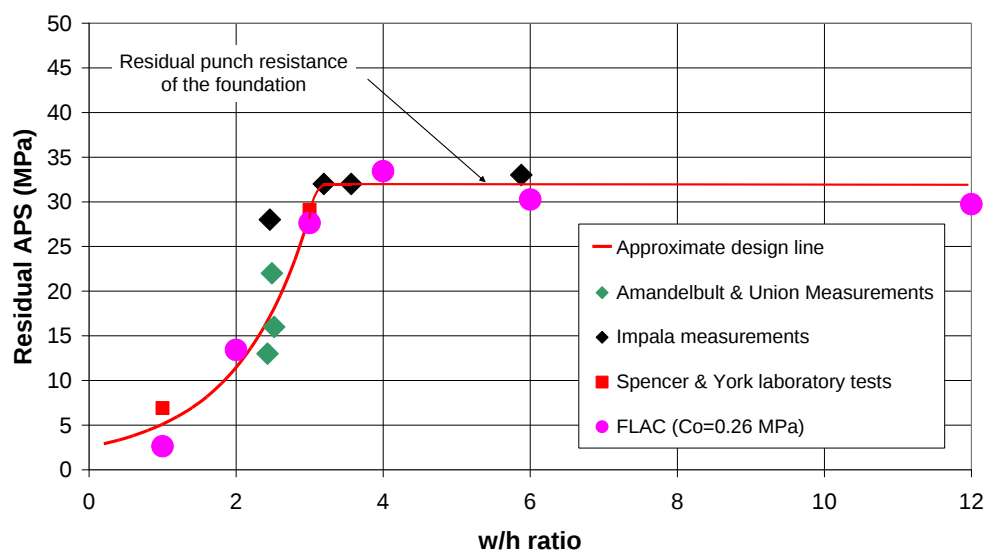


Figure 5-134 Pillar residual strength as a function of w/h ratio

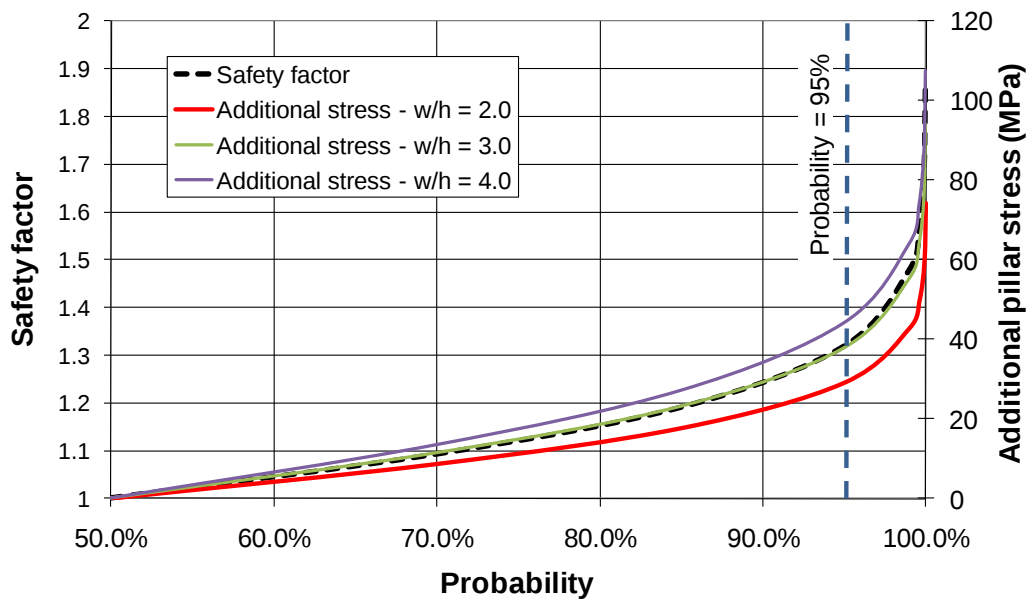


Figure 5-135 Comparison between the safety factor and the associated additional pillar stress requirements for pillars of $w/h = 2.0$, 3.0 and 4.0 , based on the linear back-fit analysis ($\log s = 0.073$)

The stoping width affects the pillar height and is often variable across a panel. For this reason pillars cut to size may have different residual strengths resulting from the stoping width adjacent to the pillar. The issue of stoping width is of particular interest when the margin between the required and supplied residual strength is small. The relationship between stoping width and pillar residual strength is provided in Figure 5-136. The relationship is based on the parameters shown in Figure 5-134, assuming a standard 3 m-wide pillar. Pillar cutting is difficult and some mines struggle to cut standard sized pillars. An analysis of residual strength verses pillar-width variability in a stoping width of 1.2 m is provided in Figure 5-137.

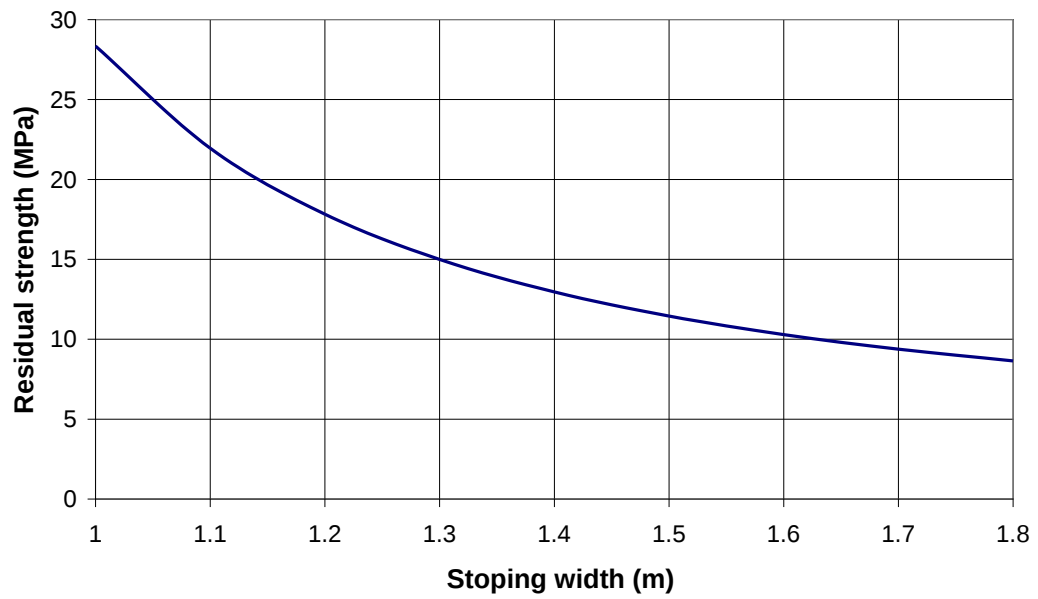


Figure 5-136 Effect of stope width on residual strength, assuming a standard 3 m-wide pillar. Salamon analytical solution with $\Phi_b = 30^\circ$ and $C_o = 1.6$ MPa

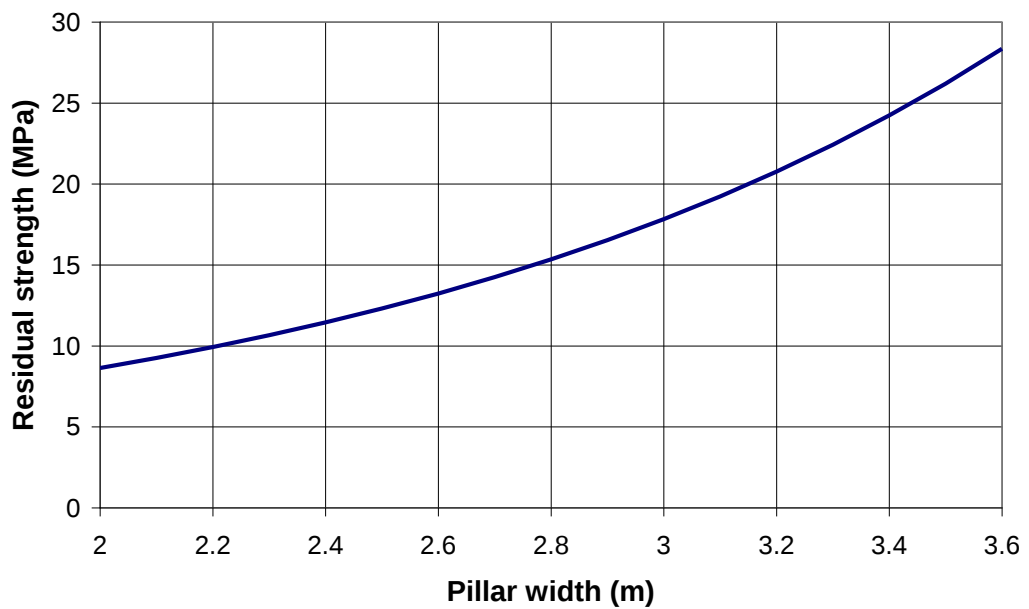


Figure 5-137 Effect of pillar width on residual strength, assuming a stope width of 1.2 m. Salamon analytical solution assuming $\Phi_b = 30^\circ$ and $C_o = 1.6$ MPa

5.7 Summary

Since the material contained in this extensive chapter is complex, and may be perceived to be confusing at times, it was decided to conclude the chapter with a detailed summary, rather than a brief one as used in the other chapters.

The strength of rock specimens and rock pillars are dependent on their end- or boundary conditions. For this reason, pillar strength should be quoted with a statement regarding the end conditions, the means of load control, loading rates etc. Chapter 4 showed that pillar failure extended into the foundation rock, indicating a complex interaction between the pillar and foundation. The study described in this chapter was based on underground measurements, with laboratory tests and numerical models being carried out to aid in the interpretation of these measurements. Most of the work was done on pillars of about 'crush'-pillar size, which means that pillar failure occurred under stable conditions and pre- and post-failure behaviour was monitored.

The influence of boundary conditions on pillars was investigated using FLAC. The modelling showed a significant difference in the behaviour of model pillars, loaded between platens in the laboratory, and the behaviour of pillars underground, with conditions that include the draping of the hanging- and footwall.

'Crush' pillars were introduced to the mining industry at Union Section by Korf (1978) to stop a serious backbreak problem where at least three to four stopes were collapsing per month. Sudden failure of the beam frequently occurred when spans reached about 60 m to 80 m, resulting in parting of the rock at the bottom contact of the Bastard Reef some 20 m above the stopes. The pillars that were introduced had a w/h ratio of about 1.5 and a length of 3 m. It was obvious from observations that the pillars crushed near the working face. Importantly, the introduction of these pillars stopped the stope collapses in the mining area where they were used.

While there is evidence that the 'crush' pillars introduced by Korf (1978) stopped the stope collapses, there is also concern that large-scale collapses could occur

at deeper levels if the residual strength of the 'crush' pillars is insufficient. Work done by Roberts et al (2004) showed that approximately 1 MPa across the stope, or 13 MPa on pillar lines spaced 30 m apart is required to stabilise the hangingwall. Little work has, however, been done to determine the residual strength of the 'crush' pillars. Thus, generally in more recent times, pillars have been cut wider than the original pillars designed by Korf (1978). Larger pillars are stronger and less likely to fail at the face, where the loading conditions are stiff. A recent series of pillar bursts with serious consequences has led to questions about the design of these pillars. Several formulae for the evaluation of peak pillar strength are available in the literature, but most of these were based on back-analysis of rock types and mining conditions that are not applicable to the Bushveld. The research carried out for this thesis thus included formulae for estimating Merensky peak pillar strength from w/h ratios.

Formulae were developed from back analyses of a database of failed and unfailed pillars from Impala Platinum to determine peak pillar strength from pillar dimensions. The results showed that the linear formula (Bieniawski and van Heerden, 1975) with appropriate strength parameters for the Merensky Reef was the most suitable for this evaluation. The resultant linear formula was successful in predicting the measured pillar strengths, both at the Impala and Union sites. It should be noted, however, that the pillars at the Amandelbult site were significantly stronger than estimated by the linear formula. The reason for the disparity may be the relatively more solid footwall below the panels at the Amandelbult site. The footwall was highly fractured at the other sites, which could have resulted in effectively higher pillars at these sites – i.e. the effective w/h ratio was smaller than that which was measured.

2D FLAC models were set up, using parameters based on underground measurements of stress and strain to verify the linear relationship between pillar strength and w/h ratio as suggested by the back analyses. The models correlated well with the back-analysis results when infinitely long pillars were assumed in the formula. Some of the findings of the numerical modelling are highlighted below:

- under conditions where the hangingwall and footwall have geomechanical properties similar to those of the pillar, an approximately linear

relationship between pillar strength and w/h ratio persists up to a w/h ratio of more than 8;

- ductile pillars are stronger than more brittle pillars;
- little or no damage in the hangingwall or footwall occurs above and below pillars with a stress of less than about three times the UCS of the foundations;
- w/h ratio strengthening effects were observed from a ratio of about 0.6;
- little strength benefit is likely to be gained by cutting pillars larger than a w/h ratio of ten;
- “squat” effects occur in hard rock at a w/h ratio of 3 where the hanging- and footwall materials are relatively strong and failure is restricted to the pillar;
- Foundation fracturing below pillars with w/h ratios greater than about 2 probably contributed to the closure in the panels;
- Except for very slender pillars, failure of hard rock pillar systems are, to a large extent, controlled by foundation fracturing and failure processes; and
- At higher w/h ratios the pillars may not be fractured throughout, but the system fails with ensuing load loss.

Seven pillars were monitored at the three instrumentation sites. Stress changes were measured with the use of 2D and 3D straincells installed in the hangingwall above the pillars. A matrix of Boussinesq equations was used to determine the optimum position for the installation of these cells to determine APS. The evaluation considered the likely peak stress evolution during pillar loading and failure. A position at 6 m above the centre of a normal-sized ‘crush’ pillar provided minimal errors at any stage of the pillar loading. However, the effects of the face and other pillars had a strong influence on the readings at this height and some assumptions about the surrounding stress influences were therefore necessary. Since these assumptions introduced significant uncertainty it was decided, in most cases, to install the cells closer to the pillar of interest. Since the stress at pillar failure was likely to cause damage to the straincell at the closer elevations, it was decided to mount these cells over the pillar edge (rather than the pillar centre). Pillar strain was calculated from closure measurements made between

1 m and 2 m from the pillar edge and, in one instance an extensometer was also installed through the pillar centre from the haulage below.

Pillar behaviour, including peak and residual strength, was established by converting the measurements made above the pillar using MinSim models and Boussinesq equation matrices. The peak pillar strength was checked with the use of the developed linear equation for peak pillar strength and stress results from detailed MinSim modelling of the mining geometry, taking into account the known face positions at the date of pillar failure. The residual pillar strength was checked by a series of field measurements conducted just over the top of the pillar. Unfortunately, the evaluation of these measurements was complicated by the influence of joints and fractures, but, in general, an inverse matrix of Boussinesq equations was used to determine the residual APS. The derived pillar profile was established using a 'best-fit' solution that necessitated some assumptions being made about the effects of the discontinuities on the measurements. For this reason the established pillar profiles can only be considered as rough estimates. All the pillars at the Amandelbult site and two of the pillars at the Impala site were influenced by ASGs. In these instances the effective w/h ratios of the pillars were adjusted to account for the ASG.

The Amandelbult site was at 600 m below surface and rated as good ground conditions (Section 2.1). Two pillars were monitored at this site. One of these pillars was instrumented with two straincells and three stress-strain curves were determined from the measurements. All three results showed an initial relatively rapid drop in stress after pillar failure, followed by a more gentle stress drop to very low stresses, and subsequent stress regeneration. In particular, one of the pillars showed a relatively large regeneration that appeared to be linked to the confinement produced by the broken rock fragments in the ASG. Once these had been cleared away, the stress regeneration halted. A FLAC model was run to determine the reason for the phenomenon. It was established that stress regeneration was possible at the measured deformation if there were minimal damage in the foundations. The instruments showed that some damage had occurred in the footwall below the pillar.

The Impala site was established at a depth of 1100 m below surface in very good rock mass conditions (Section 2.2). Three 'crush' sized pillars were instrumented and a straincell was installed at a late stage over an approximately 8 m-wide stability pillar. The stress-strain behaviours of the three 'crush' pillars were established in a manner similar to the one used at the Amandelbult site. One of the three pillars had an additional 3D cell installed above the centre of the pillar from the haulage below. The borehole in which the cell was mounted also housed an extensometer that allowed accurate evaluations of stress and strain at the centre of the pillar. Unfortunately, the hole sheared off just after pillar failure and no further measurements were possible. The peak stress over the stability pillar could not be established from the measurements because of a late installation but, since the face positions at failure were known, a MinSim model was used to establish this stress.

The pillar stress change measurements at the Impala site showed a similar drop in stress as that observed at the Amandelbult site, but once residual strength conditions were reached no further stress drops occurred. Neither did stress regeneration occur, even though the final closure was similar. Pillar stress and closure was monitored for more than two years after pillar failure at this site, and included a period of several months after mining had been completed in the stope. Interestingly, the residual strengths of the two larger 'crush' pillars and the stability pillar were similar, suggesting that the bearing capacity of the fractured foundation/s had been reached. Observations and extensometers installed in the stope hanging- and footwalls indicated that pillar punching probably occurred in the footwall. The smallest of the 'crush' pillars was about the same size as the pillars at the Amandelbult site but the residual strength was significantly greater. The reason for this anomaly is not well understood but may be the relatively small stoping height in some places on one side of the pillar.

The 3D straincell and extensometer installed above and through the centre of a pillar at the Impala site, respectively, were installed at an early stage of mining. These instruments initially recorded the stress and strain behaviour at the edge of an abutment. Unfortunately, very little of the actual pillar behaviour was monitored as the pillar failed relatively soon after formation. However, the strains measured during this short period were very much less than those indicated by

the closure meters installed in the stope at 2 m from the pillar edge. The difference between the two instruments shows the draping of the strata and inelastic behaviour.

The peak pillar strength determined by the cell over the pillar centre was similar to that of the cell located 3.4 m above the pillar edge. For this reason, the post-failure behaviour that occurred after the cable of the former cell was sheared was monitored by the latter cell.

At the Impala site, the recorded strain, during strain softening, was similar for all three pillars. Even though there was a significant difference in pillar sizes the post-failure stress-strain relationship between pillars P1 and P3 was very similar (Figure 5-80). However, a slightly different stress-strain path was followed by P2 during strain softening. The reason for this anomaly is probably the reduction in pillar size during the pillar-failure progression.

The Union site was located at a depth of about 1400 m below surface in rock mass conditions that could be described as poor (Section 2.3). A pillar was particularly cut for the purposes of instrumentation and was additional to the normal pillar configuration. In addition, it was cut on dip at the centre of a panel. The advantage of this site was that the pillar was isolated and, therefore, the unknown influences of the adjacent pillars on the stress measurements were removed. However, the disadvantage was that the pillar failed while it was a stub and, therefore, before it was properly formed. Visual observations showed that the initial failure was approximately restricted to the area of the final pillar size and it was therefore possible, with the help of numerical models, to estimate the peak pillar strength. The post-failure behaviour of the pillar was similar to that of the Amandelbult pillars, in that an initial rapid stress drop was followed by a slower drop to very low loads and, subsequently, stress regeneration occurred. Interestingly, the strain at which this regeneration occurred was the same as that for one of the pillars at the Amandelbult site. The stress-profile measurements were conducted after residual stress conditions had been reached. The results showed that the peak was off centre towards the side of the pillar protected by the abutment during pillar failure. This result indicates a possible influence of fracture density on post-failure behaviour and residual strength.

The post-peak strain-softening behaviour of all the pillars was similar in the initial unloading part of the curves. This suggests that comparatively little inelastic strain occurred during this time, as the strain measurements at the Amandelbult site were almost elastic. It is therefore concluded that the closure measurements provided reasonable estimates of the pillar behaviour, particularly in the crucial strain-softening period. A list of the measured peak and residual pillar strengths is provided together with the linear formula results for peak pillar strength in Table 5-21. Note that in all instances the average pillar residual stress remained greater than the required 13 MPa during the working life of the panel.

Table 5-21 List of all measured peak pillar strengths and residual strengths and the results of the linear formula for peak pillar strength

Site	Pillar no.	Peak strength (MPa)	Linear formula peak strength (MPa)	Residual strength (MPa)
Amandelbult	P1	320	215	20
Amandelbult	P2a	276	213	14
Amandelbult	P2b	265	213	17
Impala	P1	295	290	32
Impala	P2a	320	288	32
Impala	P2b	327	288	32
Impala	P3	263	260	28
Impala	S1	353	363	33
Union	P1	191	207	16

The only physical difference between the Impala site and the other sites that could explain the generally different post-peak behaviour at this site appears to be the presence of a persistent shallow-dipping discontinuity in the footwall. It is postulated that the structure may have influenced the fracture patterns in the footwall. Spencer and York (1999) positively linked the depth of fracturing to a similar discontinuity at another instrumentation site at Impala Platinum. However, FLAC modelling showed that a shallow-dipping discontinuity at about 1 m below the stope would have little influence on pillar behaviour if a normal friction angle

of about 30° were assumed. Possible explanations for the stress regeneration are:

- loading of the fractured pillar material by relatively less fractured foundations outside of the original dimensions of the pillar at failure, due to pillar dilation; and
- change in pillar w/h ratio due to pillar dilation and vertical shortening.

However, the same arguments apply to the Impala pillars, where stress regeneration was not observed. Interestingly, the stress regeneration on one of the Amandelbult pillars stopped and remained almost constant when the pillar reached the residual strength shown by the Impala pillars. The real mechanism for the different behaviour shown by the Impala pillars, when compared to those at the other sites, is thus not well understood and further research is recommended.

From the underground extensometer measurements, borehole camera surveys and observations, the footwall appears to have been more damaged/fractured and weakened than the hangingwall at all three sites (to a lesser or greater degree). The general dominance of footwall punching by the pillars is probably the result of a combination of factors. These probably include: mining sequences, presence of an ASG, spans, tectonic stresses, discontinuities and material properties.

The measured residual strengths and stress profiles across the pillars from the Amandelbult and Union sites were compared to analytical solutions for stress distributions in perfectly plastic materials that are yielding. The equations showed that the internal friction and cohesion of the broken rock is extremely relevant to the behaviour of a failed pillar. In addition, any horizontal stress applied at the pillar edge, e.g. support, has a similar effect. There may also be additional residual strength associated with interlocking of rough surfaces. An equivalent residual cohesion of 0.011 MPa was estimated from the effects of gravity on a pile of rock at the edge of a pillar, if a residual friction angle of 30° is assumed. The Salamon (1992) solution showed that a cohesion of about 1.6 MPa was required to simulate the magnitude of the measured residual strengths of the pillars, which suggests that the actual cohesion was significantly greater than a

simple gravitational effect. This analytical solution compared well with the results of laboratory punch tests performed by Spencer and York (1999), using the same pillar and footwall materials as the Impala site. The reason/s for the high cohesion required to simulate the residual strengths needs further investigations as the issue is fundamental to 'crush' pillar behaviour. The cohesion-uncertainty is of concern as adequate residual strength may not always be present.

It should be emphasised, that, while the laboratory tests are compared to the analytical solutions there are differences between them. For instance, the analytical solution is based on the limit equilibrium of a material that is loaded between two solid platens; in contrast, the punch test allows for damage in the foundation. This foundation damage can have serious implications as has been demonstrated for peak pillar strength. It is very plausible that residual strength is also affected by foundation damage. In addition, the formulae are based on plane strain conditions, whereas the laboratory tests were axisymmetric. This geometrical effect will also result in different relationships between w/h ratio and strength.

Despite the differences, an attempt was made to match the laboratory results with the Salamon analytical model (Salamon, 1992). A good correlation was again found when an internal friction angle of 30° and a cohesion of 1.6 MPa were selected.

Both the analytical solution and the laboratory tests showed an exponential increase in residual strength with increasing w/h ratio. Unfortunately, the w/h ratio range of the instrumented pillars at the Amandelbult and Union sites was too small to determine any meaningful comparisons with the solution. However, this solution did compare well with results from these measurements and from the pillar with a w/h ratio of about 3.2 at the Impala site. Even the results from measurements on the residual strengths of the other 'crush' pillars at the Impala site were not significantly different from the solution. However, the large stability pillar (w/h ratio ± 6) at the Impala site did not show the expected exponential increase in residual strength. In fact, the estimated residual strength was similar to the smaller pillars at the site.

The measurements at the Impala site suggested that the strength increase flattened off at higher w/h ratios. This flattening of the strength increase suggested that the bearing capacity of the fractured footwall had been exceeded, with little strength increase above a w/h ratio of about 3.2. The underground measurements, laboratory results and the analytical solutions, together, suggested an exponential increase in residual strength up to a w/h ratio of about 3.2. The Impala measurements suggested that the strength increase flattened off and no further increase in strength would occur above this ratio.

The effects of a fractured-foundation on bearing capacity were investigated using an analytical solution (Meyerhof, 1951) and FLAC models. The analytical solution suggested that an ultimate bearing capacity of 33 MPa would result from a residual cohesion of about 1.1 MPa and 0.4 MPa, assuming residual friction angles of 30° or 40°, respectively. The FLAC models indicated a cohesion of 0.26 MPa at a friction angle of 40°. The difference between the model and the analytical solution is probably because the associative flow rule was not assumed in the model. The dilation angle in the model was 10°.

A suite of FLAC models was run with the same input parameters as assumed for the verification of the linear peak pillar strength formula, but the original residual cohesion of zero was changed to 0.26 MPa. A good match was shown between the models, all the underground results (including the larger Impala pillars) and the two lower w/h ratio laboratory tests. The model results seem to confirm that the foundation bearing capacity is exceeded above a pillar w/h ratio of about 3.2, and there is, therefore, no strength benefit in larger-sized pillars. Thus the modelling, analytical solution, measurements and laboratory tests have effectively produced a design curve for 'crush' pillars. Interestingly, the change in cohesion had very little effect on the pillar peak strengths.

The main objective in 'crush' pillar design is to ensure that the residual strength of the 'crush' pillars is sufficient to arrest a backbreak. However, the peak strength also needs to be considered as these pillars need to fail stably near the advancing face. Three pillar bursts were investigated at Amandelbult to provide some insights into acceptable loading stiffnesses. (These sites were separate to the instrumentation site.) All the pillar bursts occurred at shallow depth (<600 m)

and therefore minimal inelastic deformations occurred prior to pillar failure (similar to the Amandelbult instrumentation site). In every instance, the pillars that burst were more than 10 m behind the advancing face. Since the face positions at failure were known, it was possible to model the stresses and strains at failure.

None of the pillars that were monitored at the three instrumentation sites burst. However, there were two instances where minor ejections of rock took place. In both these instances the pillars were about 5 m to 7 m behind the face. Based on the evidence of the pillar bursts and the stable pillars it was possible to determine acceptable and unacceptable loading conditions from elastic modelling. The slope of the unacceptable loading line was established at about 5.0 mm/GN. Most of the instrumented pillars failed under a post-failure loading stiffness of about 3.2 mm/GN.

Since the correlation between peak pillar strength and w/h ratio has been established, pillars can be designed to fail close enough to the face to be in a sufficiently stiff loading environment. This can be done by matching the pillar strength to the loading capacity of the strata near the advancing face. At shallow depth the loading capacity of the strata may require slender pillars that may not have sufficient residual strength to prevent a backbreak. Under such conditions panel spans may have to be reduced to ensure that the residual strength is sufficient to provide the required 1 MPa across the stope. Precondition blasting may also be an option, to ensure that the pillars fail in the stiff loading environment. These principles have been incorporated into a design flowchart (Figure 5-133).

The exponential increase in pillar strength in pillars below a w/h ratio of 3.2 suggests that these pillars are sensitive to stoping-width variability. This would be an issue of particular importance where the margin on residual strength requirements is small. A graph showing the effect of stoping width on a standard 3 m-wide pillar is provided (Figure 5-136). In addition, the effect of cutting undersized pillars is also demonstrated in a graph (Figure 5-137).

6 Conclusions

The aim of the investigation described in this thesis was to provide the South African platinum industry with a robust design procedure for Merensky ‘crush’ pillars, based primarily on underground measurements. In order to accomplish this objective it was necessary to investigate pillar behaviours in their settings of hangingwall, footwall and adjacent panels. A variety of geotechnical conditions were selected to cater for the variations in rock mass conditions across the Bushveld Complex. The discovery of nonlinear elastic conditions in the laboratory rock tests necessitated further studies of this behaviour in order to understand the rock mass behaviour. From the results of the research, the following conclusions can be drawn:

Nonlinear behaviour of rock

Nonlinear elastic rocks showed an increase in Young’s Modulus with increasing stress until the onset of sample failure. Unrealistically high Poisson’s ratios were also indicated.

The research suggests that the behaviour of nonlinear rocks is influenced by two components of strain – i.e. the solid rock around micro cracks (‘matrix’) and the closure and sliding of these cracks. Once these mechanisms were established, it was possible to develop a methodology to evaluate the stress measurements. A better understanding of the rock mass behaviour was also made possible in the affected areas. The findings of the investigations are discussed in Chapter 3.

It was found that nonlinear behaviour is more likely in polycrystalline materials such as those found in the Bushveld. Micro cracking occurs between mineral grains with different moduli during unloading, when the virgin stress conditions are sufficiently high. This behaviour related to a stress condition of about 36 MPa and 28 MPa for the horizontal and vertical stresses at the Impala site. It appears that the critical stress condition was exceeded at the Impala site because of its depth below surface and a comparatively higher k-ratio than the other two sites.

Nonlinear behaviour was also observed at the other two sites in areas where the test samples were extracted from 'high' stress conditions, such as above the pillars. In these cases, it appears that the tensile stresses that developed at the drill-bit tip probably enabled a sufficient stress drop for the micro cracks to form. However, the rock mass itself did not appear to become nonlinear at these two sites. Nonlinear rock behaviour does not appear to have affected the pillar behaviour at the Impala site either. Thus, it appears that the observed nonlinear behaviour did not influence pillar behaviour at any of the sites.

Validity of elasticity for analysis of rock mass behaviour

An extensometer was installed through the centre of a 'crush' pillar from the haulage below the reef at the Impala site. Measurements were conducted along vertical distances of 14.5 m and 4.2 m in the footwall and hangingwall, respectively. The instrument showed, that, prior to pillar failure, both the hanging- and footwall above and below the pillar were elastic, with the identical 'matrix' elastic constants as determined in the laboratory. These results suggest that a rock mass with good geotechnical conditions behaves with the same linear elastic constants as determined on a small laboratory sample. Therefore, the peak pillar strength can be reasonably estimated from elastic models. Pillar punching has been measured during pillar failure but the peak pillar strength measurements were not affected by the punching.

At all three sites, the rock mass was linearly elastic above the pillars and abutments at the positions of the stress change measurements. Again the measurements showed that the 'matrix' elastic constants, determined on small rock samples in the test laboratory, were applicable for the stress evaluation.

A series of horizontal stress measurements was conducted above the centre of the stopes at all three of the instrumentation sites. At the Impala and Union sites, the stresses in the anorthositic rock types were significantly higher than those in the pyroxenitic rocks. The measurements in both rock types could not be replicated by an elastic model with a uniform k-ratio. However, the behaviour of

the individual rock types could be simulated if separate models were run for each k-ratio. It is therefore concluded that the anorthosite and pyroxenite rock types have different k-ratios under virgin conditions. The comparatively higher horizontal stress conditions in the anorthosite compared to the pyroxenite rock types has been termed stress 'channelling'. It should be noted, however, that stress 'channelling' was not observed above the Amandelbult stope, indicating that this phenomenon may not be universal across the Bushveld.

The k-ratios in the pyroxenites at the Impala and Union sites were about 0.5. This k-ratio was also observed in the pyroxenites in the excavations below the stope at the Union site. The uniform low k-ratio in the pyroxenites at both stopes suggests that the original horizontal stresses of the Bushveld Complex, at formation, may have relaxed. However, this relaxation does not appear to have occurred in the Anorthosites, resulting in variable and comparatively higher stresses in this rock type. The uniform k-ratio in the pyroxenites was not, however, observed at the Amandelbult site and further research is required to understand the phenomenon.

Effect of boundary conditions

The influence of the stope on pillar behaviour was investigated using FLAC. The modelling showed that the draping of the hangingwall over the pillar results in high peak stresses at the edge of the pillar before failure. Failure is therefore initiated at a relatively lower APS. This early edge failure was confirmed by the underground measurements and observations.

The capacity of the interface between the pillar and the loading platen/foundation to transfer lateral/horizontal stress to the pillar results in the inner core of the pillar being confined and thus strengthened. This strengthening effect was shown by a comparison between a stope-pillar model with draping, elastic foundations and no interface, and a laboratory-pillar with elastic loading platens (no draping) and an interface friction angle of 15°. The stope-pillar is initially stiffer and subsequently more ductile than the laboratory loading environments, again resulting from an early peak stress at the edge of the pillar and the progression of failure towards

its centre. The highest peak stress at the centre of the pillar occurs after pillar failure, which is similar to the observation made by previous researchers in their underground measurements. In reality, the extent of mining around underground pillars determines the amount of hanging- and footwall drape and hence the severity of this influence.

The model results, underground measurements and observations show the importance of including the footwall and hangingwall in pillar behaviour evaluations. These evaluations should be done with realistic stope geometries as the draping of the hangingwall and footwall contribute to the overall behaviour. The investigations show that pillar behaviour cannot be simply extrapolated from laboratory tests between metal platens.

Pillar system

Failure of a pillar system, which includes the adjacent footwall and/or hangingwall rock, involves in essence a combination of three mechanisms. First, there is fracturing and crushing of the pillar itself, which often is reproduced under laboratory conditions with unrealistic boundary conditions. Then there are vertical cracks (assuming the stope is horizontal). However, footwall and/or hangingwall fracturing is not synonymous with foundation failure. Subsequently, wedge formation occurs in the form of shear fractures, which control the ultimate punch resistance. Punching is finally accommodated by horizontal dilation along Prandtl wedge-type fractures, which would add to the inelastic closure observed in a panel.

The influence of the footwall and/or hangingwall on pillar behaviour was investigated using numerical modelling, underground instrumentation and underground observations. A series of FLAC models were firstly constructed to interrogate the influence of the pillar footwall and/or hangingwall on pillar behaviour. Two sets of models were run: those represented by a more ductile material and calibrated by underground measurements of Pillar 1 at the Amandelbult site; and those represented by a more brittle material. The ultimate failure resistance for comparatively brittle and ductile materials was determined

by comparing the strengths of pillar systems where the foundations were not allowed to fail to models where failure occurred in both the pillar and the foundations. Failure was thus concentrated in the pillar where the foundations were infinitely strong and the so-called “squat” effects were demonstrated. Suites of pillars with w/h ranging from 0.5 to 10 were used in the comparison. It was found that punching could initiate at a w/h ratio of 1.2 for more ductile materials (Merensky pillars), whereas this initiation occurred at a higher w/h in more brittle materials. At smaller w/h ratios, the pillars fail by progressively crushing from the edges towards the core, but in the wider pillars additional fracturing of the hanging- and/or footwall rock is initiated. There is a disparity between the strengths of pillars with and without elastic (infinitely strong) foundations. This disparity suggests that punching is initiated once the strength exceeds 250 MPa ($\sim 3 \times \text{UCS}$). This punching was associated with different w/h ratios in the different materials. Little or no damage is likely in the foundations below these critical w/h ratios.

The closure measurements at the Amandelbult site established that the rock mass was behaving in an almost elastic manner. These measurements together with borehole camera surveys established that only minor fracturing occurred above and below the pillars at this site. A comparison between the closure measurements and elastic modelling results suggested that the fractures were confined to the area of the pillars and only extended under the stope on the up-dip side of the pillars.

The influence of pillar dilation on the immediate hangingwall above and adjacent to a pillar was established by a shallow-dipping extensometer. This extensometer was installed over a pillar and also extended over the adjacent panel at the Impala site. During pillar failure the anchor over the pillar showed dilation, while compressive deformations were measured at the same time over the panel. However, the increase in compressive stress relating to the measured average deformation above the panel half-span was small. Approximately 1 MPa was estimated using a low elastic modulus to account for the nonlinear behaviour. The distribution of this induced stress across the half-span was not, however, determined.

The horizontal stress change measurements at all three instrumentation sites were compared to the findings of elastic models. It was established that some additional compressive stress had developed in the hangingwall in a direction perpendicular to the pillar lines (or long axis of the monitored pillars) during pillar failure at all three sites. The location of the measured induced horizontal stress suggests that it probably developed as a result of vertical fracture development above the pillars. This effect of vertical fractures on horizontal stress was confirmed by FLAC modelling. Vertical fractures were also positively identified in the hangingwalls above pillars at the Impala and Amandelbult sites.

A Prandtl wedge-type fracture (associated with pillar punching) was observed in the footwall at the Impala site. The observations were made where a footwall slot had been excavated below the siding, between the pillar and the ASG. Evidence of Prandtl wedge-type fracture development was also shown by a closure profile and ride measurements at the Union site.

Pillar and stope measurements

The extensometer that was installed through the centre of a 'crush' pillar, at the Impala site, showed that slightly more inelastic deformation occurred in the hangingwall than in the footwall during and just after pillar failure. The results suggest that initially the pillar moved preferentially into the hangingwall, for a few millimetres. This initial preferential punching may be due to the comparatively lower k-ratio in the pyroxenite. However, it appears that the more ductile hangingwall material restricted failure to the area above the pillar and no fracturing occurred over the stope. The final pillar residual strength appears to have been controlled by fracturing in the footwall. Unfortunately this deformation was not measured on the extensometer as it sheared off soon after pillar failure. However, evidence of Prandtl wedge-type fractures was observed in the footwall at the site. Preferred punching of the pillars into the footwall was also indicated by bars that were cemented horizontally into the centres of two pillars at the Amandelbult site. In each of the two instances the lower bar installed near the base of the pillar rotated upwards indicating a downwards movement at the

centre of the pillar. During the same period the upper bar remained horizontal, suggesting that little pillar punching occurred in the hangingwall at this site.

Although stress measurements were not conducted in the footwalls of any of the sites, there was evidence of comparatively high stress in the immediate anorthosite footwall at the Impala site. A high degree of fracturing occurred in this material in an isolated panel, i.e. prior to pillar formation. Some form of extension fracturing (spalling) appears to have taken place when the vertical confinement was reduced as a result of mining. Similar fracturing was also observed in some strata below the Union stopes at a span of about 12 m. The very small span at which this fracturing took place is also an indication of spalling. At both sites, the fracturing resulted in inelastic closure, which may have affected the pillar deformation measurements. Borehole camera surveys showed that no fractures developed in the 7s hangingwall at the Impala site, indicating that all the measured inelastic deformation occurred in the footwall. Extensometer measurements at the Union site also showed that the greatest proportion of the inelastic closure occurred in the footwall. Some of the footwall damage may have been aggravated by the face blasts as the blast-hole configuration favoured the hangingwall, and the impact of the blast gasses was therefore greater on the footwall.

The relatively small inelastic closures measured at the Amandelbult site appear to be mainly due to pillar punching. The almost elastic closure on the down-dip side of the pillar (away from the ASG) indicates that very little fracturing occurred here. This seems to confirm the hypothesis of a relatively low horizontal stress in the anorthosite footwall at that site, and the spalling mechanism where the horizontal stresses were higher. The anorthositic rocks are also more brittle than the pyroxenites and therefore more susceptible to fracturing.

Closure measurements at the Impala site suggested that fracturing probably occurred near the face (as expected). The almost flat closure profile measured across the panel at the Impala site could not be modelled with an elastic model but can be explained by spalling. The effects of the abutment and pillars on this weakened material were also evident in the form of buckling at the panel edge, particularly at some distance back from the face. However, some buckling was

also measured towards the centre of the panel, which indicated that the spalling process may also have contributed to buckling.

Spalling may have weakened the pillars by decreasing the effective pillar w/h ratio. The phenomenon could be an explanation for the apparently stronger pillars at the Amandelbult site, where this fracturing does not appear to have occurred.

Peak pillar strength

A peak pillar-strength formula was calculated from a maximum likelihood back-analysis of a database containing stresses on failed and unfailed Merensky pillars from Impala Platinum Mine. The stresses in the database were determined by MinSim models. The analysis showed that the commonly used linear equation, with some modifications to the strength parameters and additions to account for pillar length, provided a low standard deviation for the data set. The linear relationship between w/h ratio and peak strength was confirmed by a suite of calibrated FLAC models. These models included hanging- and footwalls with the same strain-softening parameters as the pillars. The database and models show that where the hanging- and footwall materials have similar properties to the pillar, the fracturing will expand beyond the pillar itself. This punching phenomenon becomes an important aspect of the failure mechanism of the pillar system, and effectively controls the pillar peak strength at larger w/h ratios. The results of the FLAC investigation confirmed an almost linear relationship between strength and w/h ratio, between w/h ratios of about 1.2 and 8. At a w/h ratio of about 10, the punch resistance is almost at a maximum and little further increase in strength occurs with greater w/h ratios.

One stability pillar and six 'crush'-size pillars were monitored at the three instrumentation sites. Stress change was monitored by suitably placed straincells in the hangingwall. Pillar strain was estimated from closure measurements made in the stope between about 1 m and 2 m from the pillar edge. From these measurements the strength of pillars as well as pre- and post-failure pillar behaviour could be interrogated using analytical solutions. The results were checked with accurately digitised MinSim models. The generally good correlation

between the measurements at the Impala site and the pillar strength formula, suggests that the formula provides a reliable prediction of pillar strength at the Impala Platinum Mine, for the range of w/h ratios in the database. Further confirmation of the formula's reliability was shown by an excellent agreement between the linear equation and the calibrated FLAC models. A series of laboratory-simulated pillar tests were conducted with semi-realistic boundary conditions, using materials from the Impala Platinum Mine. The measured peak strengths were also closely predicted by the linear formula.

The pillar measurements at the Union site also compared well to the Impala Platinum linear equation. However, both the pillars at the Amandelbult site were stronger than predicted by the formula. These strength results and previous back-analysis of pillars at the Amandelbult Mine suggest that pillars at this mine may be stronger than at the other two mines. Further work should be done to determine the extent to which Merensky reef pillar-strengths vary across the Bushveld Complex, and whether the strength variation is influenced by footwall spalling.

Residual pillar strength

The residual pillar strengths were established from the evaluated stress-change measurements conducted in the hangingwall. These results were compared to a series of field measurements, which were analysed to produce a stress profile across the pillar. The residual strengths and profiles were subsequently compared to FLAC models, analytical solutions for perfectly plastic materials that are yielding and some laboratory punch tests. A good correlation was observed between the analytical solution and the laboratory results when a friction angle and residual cohesion of 30° and 1.6 MPa, respectively, were assumed in the equation. This exponential solution also correlated well with the underground pillar measurements up to a w/h ratio of about 3. The reasonable fit of the equation to the underground and laboratory data suggests that the range of w/h ratios up to about 3 may be adequately described by the exponential equation. No further strength increase was measured in the underground pillars for any w/h ratios greater than about 3.2. This subsequent zero strength increase could not

be simulated by the equation because the bearing capacity of the fractured foundations (hangingwall and footwall) were not included in the analysis. FLAC models that included foundations that had been damaged during pillar failure, showed that the yield capacity of the fractured foundation was exceeded at pillar w/h ratios of 3.2. These models also showed that no further strength increases occurred above pillar w/h ratios of about 3.2. The models simulated the underground residual strength measurements with surprising accuracy, when the post-failure friction angle was similar to the one used in the analytical solutions for perfectly plastic materials that are yielding. This unexpectedly high residual cohesion was also confirmed by an analytical solution for bearing capacity, assuming a residual strength of 33 MPa as measured at the Impala site. The magnitude of this cohesion could not be explained from the expected behaviour of broken rock. Further investigations are therefore recommended as the issue is fundamental to 'crush' pillar behaviour. This uncertainty is of concern because adequate residual strength may not always be present.

The results of the residual-strength investigations suggest that the bearing capacity of the fractured (residual) footwall was reached at a pillar w/h ratio of about 3. This is in contrast to the peak pillar strength, which only reaches the ultimate punch strength at w/h ratios above 10.

Design of crush pillars

Correct design of 'crush' pillars is an important consideration in mining as oversized pillars damage the foundations and may burst if failure occurs under soft loading conditions. An investigation into pillar bursts showed that these bursts occur about 10 m to 14 m behind the advancing face. This distance translated to a strata loading stiffness of about 5.0 mm/GN. None of the instrumented 'crush' pillars burst and the average unloading conditions were about 3.2 mm/GN. For this reason a strata loading stiffness of about 3.2 mm/GN can be considered safe for Merensky pillars, and this is normally achieved at less than 7 m from the face in shallow to intermediate depth operations. These findings show the importance of cutting a pillar small enough to fail close to the advancing face. In essence, the pillar strength needs to be matched to the

available load in this 3.2 mm/GN zone. Comparatively smaller pillars are required at shallower depth below surface or in lower vertical stress conditions to ensure pillar failure within the recommended strata stiffness criterion. The pillar size also affects the residual strength at pillar w/h ratios less than about 3. This may mean reduced panel sizes if the residual-strength criterion is not satisfied. It may, under certain circumstances, be necessary to precondition larger pillars to ensure failure close to the face.

The design of 'crush' pillars provides for sufficient residual strength while ensuring that the pillar is small enough to fail stably. The residual strength requirement is a support resistance of 1 MPa across the stope. This translates into a pillar stress of between 8 MPa and 13 MPa if the pillar lines are spaced 30 m apart.

An empirical methodology for 'crush' pillar design has been developed to include both the peak and residual strengths. In essence, the residual strength requirements are determined from the pillar line spacings and a pillar w/h ratio is chosen from Figure 5-134. An elastic model should be run to determine if there is sufficient load to fail the pillar within 5 m to 7 m of the lagging face. A pillar that is too large to fail within the specified distance restriction should be pre-conditioned or the pillar size and line spacings should be reduced. The concepts are incorporated into a design flowchart (Figure 5-133).

It should be emphasised that the quoted residual strengths of the pillars at the Amandelbult and Union sites reflected the average strength during the working life of the panel. However, at both these sites the strength dropped to below the acceptable limits, after mining had been completed.

References to underground measurements of peak and residual pillar strengths and pillar behaviour are sparse. The comprehensive measurement and analysis programme that was conducted for this thesis is the most extensive research into hard rock pillar behaviour in South Africa and perhaps in the world. In particular, this research has contributed significantly to the understanding of pillar behaviour in narrow tabular excavations.

7 Recommendations

The mechanism for the formation of micro cracks in the nonlinear rocks is not clear. It is recommended that further research be done to establish under what circumstances the condition occurs. More work is also required to determine the creep behaviour of nonlinear rocks in the long term. The current research did not establish whether the rocks are anelastic. This issue may have some bearing on support decisions at deeper levels. The evaluation of stress measurements conducted in the material needs further research, particularly the effects of the axial strains on doorstopper measurements. If the condition becomes more common at depth, it may be beneficial to include nonlinear behaviour in numerical models. In addition, the passage of seismic waves through this rock mass will be different and further studies are therefore suggested. The possibility of large-scale fracturing, induced by nonlinear behaviour, also needs to be investigated.

Little work has been done to determine the stress conditions in and around potholes. There is evidence that a variety of conditions may persist, with very high stresses in some of these features. A variety of different shapes also exist, which may be associated with a particular stress field.

More work is needed to determine under what conditions stress ‘channelling’ occurs and why it is not always present in the anorthosites. Stress measurements in the footwalls of stopes are recommended to determine under what conditions the observed extension fracturing occurs.

Little is understood about the behaviour of blocky ground conditions. Movements on sub-vertical and horizontal discontinuities have been observed well above the vertical and horizontal tensile zones. The effects of horizontal fractures on the vertical tensile zone have only been investigated with numerical models. The effects of the abutment, advancing face, or pillars on sub-vertical joints also need to be investigated.

The effects of gullies and sidings on pillar behaviour have only been studied with numerical models. It is recommended that suitable laboratory tests be conducted to better establish these influences.

A series of suitable laboratory tests is also recommended to confirm the peak and residual pillar strengths. In addition, tests could be conducted to determine/confirm the effect of pillar length on strength.

The peak pillar-strength formula was determined from pillars at Impala Platinum. The Amandelbult results suggest that there may be differences in strength across the Bushveld. It is therefore recommended that data be collected from each of the mining districts. The effect of footwall spalling on pillar strength also needs to be investigated.

The post-failure behaviour of the pillars at the Impala site was different from that found at the other sites. The reason for this different behaviour has not been properly established. The residual cohesion predicted by the analytical solutions and the FLAC models was high. This is an important parameter as it appears to control residual strength. However, little is known about the mechanisms behind the magnitude of this parameter.

This study concentrated on the Merensky Reef. Since the UG2 reef is comparatively more ductile, with a lower UCS but a higher angle of internal friction, the pillar behaviour is likely to be significantly different. It is therefore recommended that similar research be conducted on this reef.

8 References

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