ROCK BEHAVIOUR OF THE BUSHVELD MERENSKY REEF AND THE DESIGN OF CRUSH PILLARS

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Executive summary

Design of pillars on the tabular Merensky and UG2 reefs is empirical using experience and strength formulae derived for other ore bodies. The consequence of this uncertain methodology is to cut oversize pillars, which lowers the extraction ratio. In addition, pillars cut in the deeper levels (>600 m) are required to fail in a stable manner during or soon after being cut. These pillars are known as crush pillars and their residual strengths provide the required support resistance to prevent stope backbreaks or collapses. A recent series of pillar bursts, as a result of the pillars not fracturing and crushing at the stope face, has had serious consequences, and raised questions about the design of these pillars.

The aim of this research was to provide a proper design procedure for Merensky reef crush pillars, based primarily on underground measurements. The interaction between pillars and the rock mass was elucidated by literature studies, laboratory testing and numerical modelling. Three sites with different geotechnical conditions were selected. Nonlinear rock behaviour was measured at one of the sites. The nonlinear behaviour also occurred in laboratory tests on samples extracted from high stress conditions at the other sites, but the rock mass was not nonlinear at these sites. A methodology for determining stress from strain measured in nonlinear rock was established.

The research also established that there is an approximately linear relationship between peak pillar strength and w/h ratio at ratios between about 1.2 and 8. The so called ‘squat’ effect is not observed because pillar failure is not contained within the pillar but extends into the foundations. A linear peak pillar strength formula was established from back analyses of underground pillar failures and was confirmed by numerical modelling. Pillar behaviour was established from underground measurements on one stability pillar and six crush pillars, on which peak and residual strengths were determined. Also, stable and unstable loading conditions were established from an analysis of pillar bursts and the minimum strata stiffness for stable pillar failure was determined. This stiffness is only achieved near the advancing face and pillars that fail in the back areas are likely
to burst. For this reason, crush pillar design needs to include the peak strength as large pillars may be too strong and fail in the back area. The residual strength also needs to be considered as the load-bearing capacity of these pillars needs to satisfy the criterion of 1 MPa across the stope to prevent back-breaks. This translates into a pillar stress of about 10 MPa if the pillar lines are spaced 30 m apart. The peak and residual requirements have been included in a design chart, and the relationship between w/h ratio and residual strength is provided in a graph for easy design.

The research has contributed to the South African platinum industry by providing guidelines for Merensky crush pillar design (based on underground measurements), which was not previously available. Proper implementation of the guidelines will result in improved extraction ratios and/or safer mining using optimal pillar dimensions to ensure stable pillar crushing. A formula for peak pillar strength was also determined, for use at the Impala platinum- and other similar mines.

In particular, this research has contributed significantly to the understanding of Merensky pillar behaviour in narrow tabular excavations.
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1 General introduction

Design of pillars on the Merensky and UG2 reefs is mainly empirical using experience and strength formulae derived for other hard-rock mines. The consequence of this uncertain methodology is to cut oversize pillars, which lowers the extraction ratio. In addition, pillars cut in the deeper levels are required to fail in a stable manner during or soon after being cut. These pillars are known as crush pillars and their residual strengths provide the required support resistance to prevent backbreaks. A recent series of pillar bursts, with serious consequences, has raised questions about the design of these pillars.

This thesis describes the behaviour of pillars and the rock mass during the stoping of narrow tabular reefs on the 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. To achieve this objective seven pillars were instrumented and monitored for periods of up to three years. The instrumentation included 132 stress measurements, 160 laboratory tests, 41 closure and closure-ride stations, 8 extensometers and 9 borehole camera surveys. In addition, 183 elastic MinSim models (COMRO, 1981) and 28 FLAC models (Itasca, 1993) with strainsoftening were run.

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 and is divided into the eastern, northern and western lobes (Figure 1-1). This remarkable geological phenomenon hosts not only the majority of the world’s platinum group metals but also vast quantities of chromium and vanadium in seams parallel to the platinum ore bodies.
Figure 1-1 Map showing the western, northern and eastern lobes of the Bushveld Complex and the locations of the platinum mines

The platinum group minerals are concentrated in two extensive shallow-dipping, tabular orebodies known as the Merensky and UG2 reefs (Figure 1-2). 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.

Figure 1-2 Sketch showing the Merensky and UG2 reefs and typical shaft-access system. The detail of the on-reef stoping and pillars are presented in Figure 1-3
1.2 Stope layouts

Stope spans on the Bushveld Complex mines are determined 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 1-3.

Figure 1-3 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 1-3 is provided in Figure 1-4, where the directions of photographs ‘A’ to ‘F’ that make up the photographs in Figure 1-5 are shown.

Figure 1-4 Zoom into Figure 1-3, showing the directions of the photographs in Figure 1-5

Figure 1-5 In-stope views of: A – breast face, B – ‘crush’ pillar, C – stope below pillars, D – into stope from the gully, E – gully without sidings and F – gully with sidings

The most common mining configuration is breast mining, where the mining face is advanced on strike (Figure 1-3). 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 and are spaced about 150 m apart.

1.3 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 platinum mines at shallow to intermediate depth:

- Barrier/stability pillars, intended to remain elastic (unfailed) 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 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 at shallow depths, mainly <600 m.
- ‘Yield’ pillars, calculated to have safety factors equal to or slightly 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 deform (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 1-6 illustrate the behaviour of each type of pillar.
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 stopes > 600 m in depth 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). 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 pillar, hangingwall and footwall must be considered as components of an integrated and interacting system. To

Figure 1-6 Illustrative stress-strain behaviour of various pillar types, showing the change in behaviour for different ranges of pillar w/h ratio (after Jager and Ryder, 1999)
design these systems effectively, rock and rock mass behaviour must be understood. The mutual dependence of the stope and the pillars means that the two had to be studied together in the current research.

1.4 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 ('\(Q\)'-values between 10 and 40, (Barton, 1988)), 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 10 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 1-7. However, the thicknesses of the strata vary considerably across the Bushveld.

*Figure 1-7 Stratigraphic column showing the typical rock sequence above the Merensky Reef*
On the Merensky Reef, crush and 'yield' pillars are designed to maintain rock mass stability to the base of the overlying Bastard Reef. The pillars also reduce the demand on local stope support by restricting the height of the vertical tensile zone.

### 1.5 Aims of the research

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.
2 Description of underground experimental sites

The underground sites were classified using rock mass rating systems, i.e. RQD (Barton, 1988), Q (Barton, 1988) and N" (Watson, 2003), and are described as follows:

- Amandelbult - Shallow depth good rock mass conditions (Q = 18.3)
- Impala - Intermediate depth good rock mass conditions (Q = 50)
- Union - Intermediate-depth poor rock mass conditions (Q = 3.2)

The ratio of horizontal to vertical stress (k-ratio) varies across the Bushveld complex. The k-ratios were measured to be: unity, 1.3 and 0.5 for the Amandelbult, Impala and Union sites, respectively. All sites were mined on breast with strike spans varying between about 50 m and 150 m.

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 complicated the interpretation of stress measurements. The nonlinear behaviour was noticed at all three sites, but only the Impala site was exclusively nonlinear.
3 Nonlinear behaviour of rock

Nonlinear elastic rocks showed an increase in Young’s Modulus with increasing stress until the onset of sample failure, but was most noticeable during the first 20% of loading. Unrealistically high Poisson’s ratios were also indicated. Typical plots of strain-stress curves for the material with and without the behaviour are shown in Figure 3-1.

![Figure 3-1 Typical uniaxial test results of linear and nonlinear anorthosites from the Amandelbult and Impala mine sites, respectively](image)

The literature suggests 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. Their results showed good agreement with theoretical models, and they subsequently used the theory of non-interacting cracks to characterise micro
cracks in Charcoal Granite. In the current research, a simple 2D FLAC model of a closing crack provided a good match to the profiles of the nonlinear samples from the instrumentation sites.

The theory of non-interacting cracks suggests that the behaviour of nonlinear rocks is characterised by two components of strain – i.e. the solid rock around micro cracks (‘matrix’) and the closure and sliding of these cracks. This assumption led to the development of a methodology to carry out successful stress measurements in nonlinear rock. A better understanding of the rock mass behaviour was also facilitated in the affected areas. It should be noted that the microfracturing also reduces the strength of the rock, 210 MPa to 120 MPa in Figure 3-1.

Teufel (1983, 1989) found that seismic wave velocities could be correlated to the amount of time-dependent (anelastic) strain recovered in a core sample. This phenomenon was interpreted as being caused by the generation of micro cracks during anelastic strain recovery. The concept 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 sample and promote the strain relaxation process.

From the literature it is apparent 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 high k-ratio.
The virgin stress conditions at the other two sites appear to have been too low for these micro cracks to form in the rock mass during mining. Thus the rock mass itself did not appear to become nonlinear at these two sites. However, nonlinear behaviour was observed at these sites in areas where the test samples were extracted from ‘high’ stress situations, 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. 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.

An important result of this work is that it was possible to convert nonlinear strain measurements into reliable stresses. The methodology involved in this conversion is fully described in the thesis.
4 Rock mass behaviour

4.1 Virgin stress conditions

The k-ratios in the pyroxenites at the Impala and Union sites were ubiquitously about 0.5. This k-ratio was also observed in the pyroxenites in the tunnels below the Merensky 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 higher horizontal stresses in the anorthositic rocks has invoked the concept of tectonic stress ‘channelling’. This ‘channelling’ may be a reason for the preferred footwall fracturing in the Impala and Union stopes. The uniform and relatively low k-ratio in the pyroxenites was not, however, observed at the Amandelbult site and further research is required to fully understand the phenomenon.

4.2 Validity of elasticity for analysis of rock mass behaviour

A vertical extensometer was installed through the centre of a crush pillar from the footwall haulage below the reef at the Impala site (Figure 4-1). Deformation was measured in the 14.5 m of footwall and 4.2 m of hangingwall penetrated. The instrument showed, that, prior to pillar failure, both the hangingwall and footwall of the pillar were elastic (Figure 4-2 and Figure 4-3), with the ‘matrix’ elastic constants as determined in the laboratory. These results suggest that a rock mass with good geotechnical conditions has the same linear elastic constants as determined on a small laboratory sample. Pillar punching was measured during pillar failure but neither the stress measurements nor the elastic model results of peak pillar strength were affected by the punching.
Figure 4-1 Impala site: section along the extensometer hole, drilled through the centre of Pillar P2 (not drawn to scale)

Figure 4-2 Impala site: measured and elastic hangingwall deformations above the centre of Pillar P2, between the pillar top contact and 4.18 m above the pillar. $E = 83$ GPa and $\nu = 0.32$ were used in the model. Negative face advances represent the face position behind the instrument. Positive deformation represents compression.
Figure 4-3 Impala site: footwall extensometer below the centre of Pillar P2, between the pillar bottom contact and 14.5 m below the pillar. Measured and elastic deformations plotted against the advance of the 8s face. Negative face advances represent the face position behind the instrument. Positive deformation represents compression

At all three sites, the rock mass was linearly elastic above the pillars and abutments at the positions of the stress change measurements. However, nonlinear behaviour was measured after pillar failure at the Impala site once the stress had dropped low enough.

4.3 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. Fracturing is therefore initiated at a relatively low 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 2D FLAC modelling and is described in the thesis.
A realistic model allows the fracturing or damage to expand beyond the pillar, into its foundations. 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 literature, 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.

4.4 Boundary conditions at the Amandelbult site

The closure measurements at the Amandelbult site established that the rock mass over the stope 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 showed that the fractures only influenced convergence on the immediate up-dip side of the pillars (Figure 4-4).

![Graph](image.png)

Figure 4-4 Amandelbult site: measured closure and modelled elastic convergence. 1 m up-dip and 2 m down-dip of P2
The fact that very little foundation fracturing took place at the Amandelbult site makes this site ideal for comparison to analytical solutions that do not consider the affects of the foundations on pillar behaviour. Thus the residual stress profiles measured over Pillar P1, at this site, will be compared to relevant analytical solutions for residual pillar strength in the following chapter. The almost elastic closure that occurred at this site also made it possible to accurately measure pillar deformations without the errors associated with foundation fracturing. The measurements of pillar stress and strain at this site were therefore used to calibrate FLAC models. These models are described in the following chapter.
5 Pillar behaviour

5.1 Pillar system

The influence of the footwall and/or hangingwall on pillar behaviour was investigated using numerical modelling, underground instrumentation, underground observations and laboratory tests. A series of FLAC models were 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 from Pillar P1 at the Amandelbult site; and those represented by a more brittle material. The input parameters for these models are shown in Figure 5-1. The hangingwalls and footwalls were included with the same strain-softening parameters as the pillars. 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.

![FLAC model properties](image)

**Figure 5-1 FLAC model properties (the more ductile model was calibrated from underground measurements)**

The ultimate foundation failure resistance for 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. The results of the comparison are shown in Figure 5-2, the infinitely strong foundation results being denoted by “Pillar brittle” and “Pillar ductile”. Failure was concentrated in the pillar where the foundations were infinitely strong and the so-called “squat” effects were demonstrated. Suites of pillars with w/h ratios 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 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 hangingwall 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 stress exceeds 250 MPa (~3 x UCS). Little or no damage is likely in the foundations below these critical w/h ratios (w/h < 1.3 for Merensky pillars).

![Figure 5-2 FLAC: 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](image)

The results of the FLAC investigation suggested that a linear relationship exists 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 (Figure 5-2).
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. This fracturing occurs at small w/h ratios and often is reproduced under laboratory conditions even with unrealistic boundary conditions. Secondly vertical “Herzian” cracks (Hertz, 1896) form in the foundation (Figure 5-3 - left). However, this type of footwall and/or hangingwall fracturing is not synonymous with foundation failure. Subsequently, wedges form by shear fracturing (Figure 5-3 - right). These shear fractures and wedges control the ultimate punch resistance. Finally, punching is accommodated by horizontal dilation along Prandtl wedge-type fractures (Prandtl, 1921), which would add to the inelastic closure observed in a panel. The latter-type fractures only occur under/over pillars with w/h ratio >3.

**Figure 5-3 FLAC: failure distribution, using dense mesh and ductile material; w/h =2.0 (left) and 5.0 (right) (double symmetry)**

A Prandtl wedge-type fracture was observed at the Impala site in a footwall slot that had been cut between the pillar and the gully (Figure 5-4). Evidence of Prandtl wedge-type fracture development was also shown by measurements of closure and ride that demonstrated footwall heave at the Union site.
Figure 5-4 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.

5.2 Pillar and stope measurements

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 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 models, laboratory tests and analytical solutions. The results were checked with MinSim models using a small grid size.

The influence of pillar dilation on the immediate hangingwall above and adjacent to a pillar was established by a shallow-dipping extensometer (Figure 5-5). This extensometer was installed over a pillar (P1) and also extended over the adjacent panel at the Impala site. During pillar failure the anchors over the pillar showed
dilation (Figure 5-6), while compressive deformations were measured at the same time over the panel (Figure 5-7). 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 determined however.

**Figure 5-5 Impala site: section along the shallow-dipping extensometer borehole, drilled over the top of Pillar P1 and Panel 8s (not drawn to scale)**

![Figure 5-5](image)

**Figure 5-6 Impala site: horizontal dilation in the hangingwall across the top of Pillar P1, about 0.9 m above the pillar. Compression is positive**

![Figure 5-6](image)
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 (Figure 5-8) could not be reproduced with an elastic model but can be explained by horizontal fracturing of the footwall anorthosites. 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. Ryder and Jager (2002) suggest that buckling is possible in high-stress environments, where long narrow slabs have developed.
**Figure 5-8** Impala site: comparison between the measured closure and elastic convergence from a MinSim model for an isolated panel. Measurements and model initiate at a face position of 6.5 m from the instrument line

The flat fracturing may have weakened the pillars at the Impala and Union sites by increasing the effective pillar height. The lack of flat fracturing at the Amandelbult site could explain why the pillars were slightly stronger than at the other two sites.

Measurements carried out on Pillar P1 at Impala are plotted as a stress-strain curve in Figure 5-9. The dimensions of this pillar were 6.8 m x 5.6 m (length x width) and an average height of 1.6 m. Similar curves were produced from measurements taken on all seven pillars.
At the Impala P1 site, a 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 when the face had advanced 2.7 m past the pillar (Figure 5-10). Modelling showed that the effects of the surrounding pillars and face abutment on the measurement point at failure were minimal. 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 (Figure 5-11). Nonlinear behaviour was only measured when the stress at the measurement point dropped to about 6 MPa. The conversion of the measurements to pillar stresses was made with the help of appropriate elastic models. The section of the curve to the right of the sudden drop in stress (dotted region in Figure 5-9) was evaluated using the techniques developed during the research, described in the thesis, for evaluating nonlinear behaviour.
Figure 5-10 Sketch showing Impala Pillar P1 in relation to the 8s panel face abutment and other pillars at failure, plan view

Figure 5-11 Impala site: section showing the straincell position above P1 (not drawn to scale)

The P1 analysis suggests pillar peak and residual strengths of 295 MPa and 32 MPa, respectively. The strain was calculated from closure measurements conducted in the stope about 1 m down-dip of the pillar.
5.3 Peak pillar strength

A peak pillar-strength formula was established from a maximum likelihood back-analysis of a database containing stresses of failed and unfailed Merensky pillars from Impala Platinum Mine. The stresses in the database were determined from small grid-size MinSim models. The analysis showed that the commonly used linear equation, with some modifications to the strength parameters and additions developed by Ryder et al (2005) to account for pillar length (L), provided a low standard deviation (1.2 MPa) to the data set. The original form of this solution is provided in Equation 5-1.

\[
PS = K \left[ \frac{1 + a}{1 + \frac{aw}{L}} \right] \left[ b + (1 - b) \frac{w}{h_e} \right]
\]  

5-1

“\( h_e \)” represents the effective height of a pillar whose one side is adjacent to a gully. The parameter is calculated using Equation 5-2 (Roberts et al, 2002).

Where an adequate siding has been cut between the pillar and the gully, \( h_e = h \).

\[
h_e \approx \left[ 1 + 0.2692\left(\frac{w}{h}\right)^{0.08} \right] h
\]  

5-2

The back-fit values for Equation 5-1 are shown in Equation 5-3.

\[
PS = 136 \left[ \frac{1.27}{1 + \frac{0.27w}{L}} \right] \left[ 0.59 + 0.4 \frac{w}{h_e} \right]
\]  

5-3

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 (0.59) is similar to values obtained by Bieniawski and van Heerden (1975) for large in situ South African coal specimens (b = 0.64). The determined in situ cube strength (K\textsubscript{i}) appears slightly high for Merensky Reef as the laboratory UCS determined from anorthosite cylinders, with w/h ratios of 0.3, was only about 90 MPa for specimens from the Impala site. However, these tests were affected by microfracturing and the rock mass in and around the pillars was
The linear relationship between w/h ratio and peak strength was confirmed by the suite of calibrated FLAC models described in Section 5.1.

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):

- >3: Pillar presumed failed, date/geometry at failure not accurately known;
- 3: Pillar definitely failed (or burst), date/geometry at failure known; and
- <3: Pillar presumed not failed, date/geometry known.

A comparison between the modelled APS and calculated strength values using Equation 5-3 is shown in Figure 5-12. The figure shows a good separation between failed (CC>3) and unfailed (CC<3) pillars, with a correspondingly low evaluated standard deviation. The good fit is particularly evident from the CC = 3 cases in which pillar failure strength was obtained with confidence.

![Figure 5-12 Plot of failed and unfailed pillars with the linear formula compared to the modelled strengths (Equation 5-3)](image)

In Figure 5-13 the calibrated FLAC pillar analysis is compared to Equation 5-3 and some specially prepared laboratory tests performed by Spencer and York (1999). The laboratory tests were carried out using a cylindrical punch of 25 mm diameter and a foundation cylinder of 80 mm for the diameter and the height. 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 (steel to rock) was unrealistic and this parameter 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 therefore will be greater than 50% of the actual w/h ratio of the punch, but the effective ratio cannot be quantified with any certainty. In addition, the shape of the pillar was different from 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 similarity is shown by the curves Figure 5-13. In particular, a good correlation is shown between Equation 5-3 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 5-13 Comparison of the strengths predicted from the database (Linear back-analysis), FLAC modelling (system brittle and system ductile) and laboratory tests performed by Spencer and York (1999)](image-url)
The generally good correlation between the underground measurements at the Impala and Union sites and the pillar strength formula (Figure 5-14), suggests that the formula provides a reliable prediction of peak pillar strength at these sites, for the range of w/h ratios in the database.

![Figure 5-14 Comparison between the measured pillar strengths and the 'back-fit' pillar strengths, using the linear pillar-strength formula (Equation 5-3)](image)

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 site 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 the flat fracturing in the footwall or by geological factors such as the so called iron enrichment phenomenon.

### 5.4 Residual pillar strength

The residual pillar strengths were established from the stress-change measurements conducted in the hangingwall. These results were compared to a
series of field measurements, which were analysed using an inverse matrix of Bousinesq equations to produce a stress profile across the pillar. A detailed description of the process is provided in the thesis. The residual strengths and profiles were subsequently compared to analytical solutions for perfectly plastic materials that are yielding, the Spencer and York (1999) punch tests and FLAC models.

Salamon (1992) derived relatively complicated expressions to describe the stress distribution in a plastic pillar, that allow for a non-uniform stress distribution in vertical slices (Equation 5-4). These expressions are, however, more realistic than a similar solution suggested by Barron (1983).

\[
APS_p = \frac{2h \times 0.8}{2.5w} \frac{C_o}{0.5} \left[ e^{\frac{2w}{3h}} - 1 \right]
\]

5-4

Where:

\( C_o = \) Cohesion

A friction angle of \( \Phi_b = 30^\circ \) was suggested by the exponent in the \( w/h \) ratio relationship with residual strengths, as shown by the Spencer and York (1999) laboratory tests. This angle was also confirmed by tilt tests performed on anorthosites. The residual cohesion can be thought of as a frictional effect due to gravity, which leads to values in the low kPa range. The residual strength could also be enhanced by 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 (Ryder and Jager, 2002). However, a cohesion of 1.6 MPa provided a better back-fit of Equation 5-4 to the laboratory results.

The curves for cohesions of 0.011 and 1.6, the underground measurements and the laboratory results are plotted in Figure 5-15. For completeness the results of the FLAC models are also included in the figure. These models were conducted using the same strain-softening parameters as in the pillar-behaviour simulations (Figure 5-1), but assuming a final \( C_o = 0.3 \) MPa for the pillars and foundations. The residual strengths of the model and pillars in Figure 5-15 were determined using the small strain option in FLAC.
Figure 5-15 Pillar w/h ratio-strengthening effects on residual APS. The underground 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 5-4 fitted the underground measurements at the Amandelbult and Union sites as well as the laboratory tests when a residual cohesion of 1.6 MPa was assumed in the equation. However, the underground data from these sites is clustered within a very small range of w/h ratios.

In order to check the validity of the Salamon equation for determining the effects of pillar w/h ratio on strength, Equation 5-4 was differentiated and compared to the measured stress distribution across Amandelbult Pillar P1. The differentiated equation is shown in Equation 5-5, and provides the vertical stress distribution, horizontally across the centre of a pillar ($h$), assuming $\phi_b = 30^\circ$ and a cohesion of 1.6.

\[
S_{yy} = 3.2 \left( 0.8 \times e^{\left( \frac{2.54}{28} \right)} \right) \tag{5-5}
\]

The comparison between the solution and the measured stress profile for Amandelbult P1 is provided in Figure 5-16. 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. However, the inelastic deformations at the Amandelbult site were small, and the good correlation between the solution and the measurements therefore suggest that the assumptions for cohesion and friction are approximately correct.

![Graph showing stress profile](image)

**Figure 5-16 Comparison between the Salamon formula (Equation 5-5), assuming Φₘ = 30° and Co = 1.6, and the measured Amandelbult P1 stress profile**

In Equation 5-5, a high w/h ratio would be associated with vertical stresses that are higher than the punching resistance of the foundation (hangingwall or footwall). This is an explanation why the larger w/h ratio pillars may not be well represented by the analytical solutions shown in Figure 5-15.

The peak strengths of the Spencer and York (1999) laboratory tests had almost a linear relationship with w/h ratio, as discussed in Section 5.3. However, the residual strength relationship was exponential (Figure 5-17).
The initially higher rate of increase of peak strength with w/h ratio in Figure 5-17 was also shown by the FLAC modelling. 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 a large increase in the depth of damage into the foundation when w/h ratios were increased from one to three in the laboratory tests (Figure 5-17). The increase in the depth of damage between w/h ratios of three and five is smaller, possibly as a result of the mobilisation of Prandtl wedge-type fractures.

It should be emphasised that there are fundamental differences between the 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 significant implications for peak pillar strength as has been described in Section 5.2. 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 5-4). Since the Salamon equation overestimates the stress around the core of the pillar, 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 5-4) need to be adjusted. A sensitivity analysis showed that the friction angle and the residual cohesion could vary between 25° and 31° and 1.6 MPa and 0.7 MPa, respectively, to match the laboratory results with the equation.

The magnitude of the calibrated internal friction angle (30°) seems realistic, especially since the actual material friction angle can be expected to be slightly 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 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. The residual strength is probably influenced by a combination of the following factors:

- Interlocking blocks;
- Block size;
- Failure violence;
- Peak strength; and
- Residual cohesion.

These factors may not be constant.
The uncertainty of retaining the relatively high residual cohesion is of concern as a drop in the cohesion may result in a drop in residual strength. This possible drop in cohesion was suggested by the time dependant decrease in stress that was measured over a period of years after mining had been completed at the Union site.

Neither the analytical nor the laboratory results matched the flat, linear relationship between pillar residual strength and w/h ratio as measured underground at the Impala site (Figure 5-15). Since the pillars at this site represent mainly the higher w/h ratios, the effects of the fractured foundation bearing capacity were investigated using FLAC modelling and an analytical solution for bearing capacity.

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 5-15 suggest that an increase in residual strength with w/h ratio only occurs up to pillar w/h ratios 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 5-6 (Meyerhof, 1951). The dilation and friction angles are assumed to be the same in the equation (associative flow rule).

$$BC = \left[ \left( e^{\pi \tan \phi} \tan \left( 45 + \frac{\phi}{2} \right) \right) - 1 \right] \cot \phi \times C_0 \quad 5-6$$

Bearing capacity has been plotted as a function of cohesion for friction angles of $30^\circ$ and $40^\circ$ in Figure 5-18. The ultimate bearing capacity of the pillars, as shown in Figure 5-15, appears to be about 33 MPa. This suggests cohesions of about 1.1 MPa or 0.4 MPa for friction angles of $30^\circ$ or $40^\circ$, respectively. The FLAC models indicated a cohesion of 0.3 MPa at a friction angle of $40^\circ$. 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 $10^\circ$.  

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The good agreement between Equation 5-4 and the residual strength from the laboratory results as given in Figure 5-15 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 5.3. The residual strength results were, however, different from the underground and laboratory results,

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 1.0 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 5-19. 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 5-15). 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 ratio of about 10 as discussed in Section 5.3. 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 the former and latter models on the peak pillar strength.

![Figure 5-19 Stress-strain curves of pillar-systems with different pillar w/h ratios. Generated by FLAC, assuming a constant pillar width and varying the pillar height. Residual cohesion = 1.0 MPa](image)

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 0.3 MPa was
assumed in the model. These FLAC results have been included in Figure 5-15. Finally, the friction angle of 40°, which may be applicable to the pre-failed pillar, may be too high for the failed pillars.

The results shown in Figure 5-19 suggest that the currently accepted pillar behaviour, as shown in Figure 1-6 does not apply to a pillar system that includes foundations with similar or weaker strength properties than the pillar. Both the models and underground measurements showed that Merensky pillars with w/h ratios greater than five can fail under sufficiently high loading conditions. The conditions shown in Figure 1-6 only hold if relatively low strains are considered, such as indicated by the dashed line in Figure 5-19.

5.5 Design of crush pillars

The design of crush pillars should provide for sufficient residual strength while ensuring that the pillar is small enough to fail stably. The residual strength requirement is for a minimum support resistance of 1 MPa across the stope (Roberts et al, 2005). This translates into a pillar stress of about 10 MPa if the pillar lines are spaced 30 m apart.

Crush pillars that are designed or cut too large are a potential safety hazard as oversized pillars are too strong and may burst if failure occurs under soft loading conditions. At shallower depths violent failures of over-sized pillars generally occur at a significant distance behind the working area with little risk. However, as mining proceeds deeper, violent failures occur closer to the working area. An investigation into pillar bursts showed that these bursts occur from about 10 m to 14 m behind the advancing face. MinSim models were used to determine the strata loading stiffness at 10 m behind the face. A loading stiffness of about 5.0 mm/GN (line “A” in Figure 5-20) was determined from the models. None of the instrumented crush pillars burst and the average unloading conditions were about 3.2 mm/GN (Line “B” in Figure 5-20). A semi-stable pillar failure occurred within 7 m from the lagging face. 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 within 5 m of 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 5 m zone, where the loading stiffness is sufficiently high. Smaller pillars are therefore required at shallower depths 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 spans 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.

Figure 5-20 Acceptable (B) and unacceptable (A) stiffness of the surrounding strata for stable crush pillar design

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, bearing in mind the 1 MPa support resistance requirement across the stope. The pillar w/h ratio is established from Figure 5-21. An elastic model should be run to determine if there is sufficient load to fail the pillar within 5 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 w/h ratio and line spacings should be reduced. The concepts are incorporated into a design flowchart (Figure 5-22).
Figure 5-21 Pillar residual strength as a function of w/h ratio

- Determine residual strength requirements
- Optimum pillar dimensions from Figure 5-21
- Determine peak pillar strength from Equation 5-3
- Determine the available load from an elastic model
  - Change panel span or pillar size
  - Stress sufficient to fail pillar near face
  - Preconditioning
  - Stop

Figure 5-22 Flowchart for Merensky crush pillar design
The design line shown in Figure 5-21 is divided into two sections. To the left of the dashed line, the design profile is described by the Salamon equation (5-4), which was calibrated by the Spencer and York (1999) laboratory tests. The remarkable fit to the FLAC results and the underground measurements lends credence to the exponential profile of this line for pillars with w/h ratios up to about three. Both the underground measurements and the FLAC modelling show a flat profile to the right of the dashed line. The design line in this region of the graph assumes a fractured foundation with a bearing capacity of 33 MPa. There is therefore no strength benefit of cutting pillars with w/h ratios greater than about 3.2.
6 Conclusions

In the literature there are few references to underground measurements of peak and residual pillar strengths and pillar behaviour. 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 Merensky pillar behaviour in narrow tabular excavations.

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

Laboratory tests performed on nonlinear elastic rock materials showed an initial increase in Young's Modulus with increasing stress. Unrealistically high Poisson's ratios were also indicated. The research suggests that the behaviour of these rocks is influenced by closure and sliding of microcracks. Laboratory results were therefore controlled by two components of strain, i.e. a linear elastic ‘matrix’ and the behaviour of the microcracks. In the context of Rock Mechanics, the concept of two strain components was used to develop a methodology to evaluate the stress measurements conducted in this material.

Rock mass behaviour

The results of extensometers showed the validity of using elastic theory for the analysis of the underground measurements conducted in unfractured rock. This extensometer was installed through the centre of a pillar from the haulage below.
Nonlinear elastic behaviour was observed in laboratory tests from all three of the instrumentation sites. However, at the Amandelbult and Union sites, only samples retrieved from high stress areas showed the phenomenon. At these sites, the test samples were influenced by the tensile stresses at the drill-tip during drilling and the rock mass did not become nonlinear. However, the rock mass at the Impala site was nonlinear in areas where the stress had dropped below 6 MPa.

The concept of stress ‘channelling’ in the anorthositic rocks was suggested by measurements conducted at the Impala and Union sites. The comparatively higher horizontal stress in this rock type is thought to be a reason for the footwall fracturing at these two sites. However, the phenomenon was not measured at the Amandelbult site and further research is therefore required to verify the concept. This fracturing may have influenced the effective pillar w/h ratios and could be an explanation for the stronger pillars at the Amandelbult site.

FLAC models were used to clarify the interrelationship between stopes and pillars. It was found that the draping of the hangingwall and footwall over the pillars results in premature edge failure of pillars and subsequently also effects their brittleness. In addition, where the foundations are of similar strength to the pillar, failure may not be contained in the pillar only but in larger w/h ratio pillars will extend into the foundations.

In essence, the failure behaviour of a pillar system involves a combination of three mechanisms. Initially, there is fracturing and crushing of the pillar itself, which occurs at small w/h ratios. Subsequently vertical “Herzian” cracks form in the foundation. However, this footwall and/or hangingwall fracturing is not synonymous with foundation failure. Finally, shear fracturing and wedge formation occurs below larger w/h ratio pillars which controls the ultimate punch resistance. Punching is accommodated by horizontal dilation along Prandtl wedge-type fractures, that would add to the inelastic closure observed in a panel.

Vertical fractures were observed in the hangingwall above pillars at the Amandelbult and Impala sites. The dilation that occurred during the formation of these fractures was also measured by a sub-horizontal extensometer installed
over a pillar at the Impala site. Prandtl wedge-type fractures were observed in a

cutting that was blasted between the gully and a pillar at the Impala site.

The pillars at the Amandelbult site were ideal for calibrating numerical models as
little inelastic deformation occurred where the closure measurements were made.
It was therefore possible to measure the strain associated with pillar failure
without the errors associated with footwall or hangingwall fracturing.

Pillar behaviour

Strain softening parameters were determined from the pillar measurements at
Amandelbult. A series of FLAC models were run to determine the effect of w/h
ratio on peak and residual strength. A sensitivity analysis was done on brittleness
and the effects of infinitely strong, elastic foundations were investigated. The
results showed that punching initiated at \( \sim 3 \times \text{UCS} \). The w/h ratio at which
punching initiated was influenced by material brittleness. Little or no damage is
likely in the foundations below pillars with low w/h ratios. In the more ductile
(calibrated) models, the initial rate of strength increase with w/h ratio appears to
have been steeper until the initiation of foundation failure. An approximately linear
relationship between pillar strength and w/h ratio was shown in the range from
one to about eight.

A linear relationship between peak pillar strength and w/h ratio was also
suggested by the results of a Maximum Likelihood evaluation of a database of
failed and unfailed Merensky pillars. An empirical equation for peak pillar strength
was derived from the analysis of the Impala pillars. Infinitely long pillars were
used in the equation to allow comparison with the calibrated FLAC models, and a
good correlation was observed. A series of appropriate laboratory tests also
correlated well with the model results.

Underground stress change measurements were conducted at optimum heights
above seven pillars with a range of w/h ratios at the three sites. Both the peak
and residual strengths were determined from these measurements. The peak
measurements were checked using small grid-size MinSim models. The APS of
the pillars at the Impala and Union sites correlated well with the derived formula for peak pillar strength.

The measured pillar residual strengths were compared to a set of suitably prepared laboratory tests, analytical solutions for perfectly plastic materials that are yielding and FLAC models. The solutions and the laboratory tests suggested an exponential relationship between strength and w/h ratio for pillars ranging up to a w/h ratio of 3.2. However, no significant increase in pillar strength was measured in pillars with w/h ratios greater than 3.2, and this limit was also confirmed by FLAC models. The FLAC models also correlated well with the laboratory tests and pillar measurements for w/h ratios less than 3.2. The pillar measurements and FLAC models suggest that the bearing capacity of the fractured foundations is 33 MPa and no further increase in strength can be expected.

The internal friction angle for the crushed pillars was established to be between 30° and 40°. The laboratory tests, analytical solutions and the FLAC models showed unexpectedly high values of cohesion for these friction angles. An analytical solution for bearing capacity confirmed the high cohesions, ranging between 1.1 MPa and 0.4 MPa for the 30° to 40° friction angles. This range of cohesions was established assuming a bearing capacity of 33 MPa in the solution. The uncertainty of retaining the high residual cohesion is of concern as a drop in this parameter will result in a drop in the residual strength of the pillar. Further research is therefore recommended to determine the mechanism/reasons for this high cohesion.

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.

**Crush pillar design**

The established relationships of peak and residual pillar strengths with w/h ratio were used to develop a design methodology for Merensky crush pillars. The
literature suggests that a stress of 1 MPa across the panel is required to avoid a backbreak. This translates into a residual stress of 10 MPa if the pillar lines are spaced 30 m apart. The residual strength requirements determine the pillar w/h ratio. Pillars cut too wide for the loading environment may fail violently under a soft loading environment. It was found that, at shallow to intermediate depth below surface, violent failure did not occur when pillars failed within 5 m of the lagging face. The loading stiffness of the surrounding strata is about 3.2 mm/GN in this stable loading environment. Should the loading capacity of the strata be insufficient to fail the pillars within 5 m of the face, either lower w/h ratio pillars are required or preconditioning could be considered. Smaller pillars should be designed in the context of the 1 MPa support requirements, which may mean a reduction in the panel span.

As a consequence of this work on monitoring the stopes and pillars, a better understanding of rock mass behaviour around stopes and pillars has been gained. It is realised that the design of pillars must be considered as a system including the pillars, foundations and the stope. The design is based on the minimum of 1 MPa support resistance across the stope which in turn defines the dimensions and spacings of pillars. A flowchart was developed to enable easy crush pillar design (Figure 5-22).

The research has contributed to the South African platinum industry by providing guidelines for Merensky crush pillar design (based on underground measurements), which was not previously available. Proper implementation of the guidelines will result in improved extraction ratios and/or safety using optimal pillar dimensions to ensure stable pillar crushing. In addition, a formula for peak pillar strength was established for use at the Impala platinum- and other similar mines. A comparable study of UG2 behaviour would be required for design of crush pillars on this reef.
7 References


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