The stress-strain behaviour of in-stope pillars in the Bushveld Platinum deposits in South Africa

by B.P. Watson*, M.K.C. Roberts*, M.M. Nkwana*, J. Kuijpers*, and L. van Aswegen†

Transaction Paper

Synopsis

In South Africa, the Bushveld platinum group metal deposits are unique in that they are the largest platinum group metal deposits in the world. These deposits occur as two distinct stratiform tabular orebodies and strike for many hundreds of kilometres. Mining is extensive, with depths ranging from close-to-surface to 2 000 m. The mining method is a variation of planar open stoping. Pillars are widely used to support the open stopes. Crush pillars are commonly used in this role, where the residual pillar strength provides the required support resistance to stabilize the stoping excavations. This paper describes the direct measurement of stress within these crush pillars, and describes the stress–strain behaviour of these pillars. These findings indicate that the measured pillar failure stress is too high. The implications of these findings are that roof damage and pillar bursting could occur, and some examples of this type of damage are shown. The paper concludes that this problem could be mitigated if the three dimensional pillar geometry were modified and the pillars were cut smaller.

Introduction

This paper was published in the proceedings of *GoldenRocks, 41st U.S. symposium on Rock Mechanics '50 years of Rock Mechanics—landmarks and future challenges*'. The American Rock Mechanics Association (ARMA) is acknowledged as being the copyright holder.

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 (see Figure 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 orebodies some hundreds of metres in the footwall and hangingwall respectively. The platinum group metals are concentrated in two dipping planar orebodies known as the Merensky Reef, a mineralized pegmatoidal pyroxenite 0.7 m to 1.4 m thick, and, underlying this, the UG2 Reef comprising one or more chromitite seams of similar thickness. The strata generally dip toward the centre of the complex at 8° to 15°. The horizontal to vertical stress ratio (*k* ratio) varies from about 0.8 to over 2.5 and locally can cause severe strata control problems, particularly in tunnels. The depth of mining ranges from outcrop to 2 300 m.

In the mining depth range from surface to about 1 400 m there is a vertical tensile zone in the stope hangingwall. If a sufficiently large mining span is achieved, or the stope abuts a geological feature, a large volume of hangingwall rock can become unstable, resulting in a stope collapse or, colloquially, a 'backbreak'. In order to prevent these backbreaks a high support resistance support system is required. This is universally achieved by the use of small in-stope chain pillars orientated either on strike for breast mining (see Figure 2) or on dip for up-or-down dip mining.

The pillars are known as crush pillars and they are required to fail in a stable manner soon after being cut. The residual strength of the pillars provides the required support resistance to prevent backbreaks and keep the stope hangingwall stable. A recent series of pillar bursts, with serious consequences, have raised questions about the pillar sizes and stability with time. These and other technical issues could be resolved by measuring the pillar stress and determining how this stress varies with time. The paper describes an instrumentation site and suggests how the results could be employed to reduce the risk of pillar bursts.

* *CSIR, Johannesburg, South Africa.*

† *Amandelbult, A Division of Anglo Platinum, North West province, South Afrca.*

 SA ISSN 0038–223X/3.00 + 0.00. Paper received Nov. 2006; revised paper received Mar. 2007.

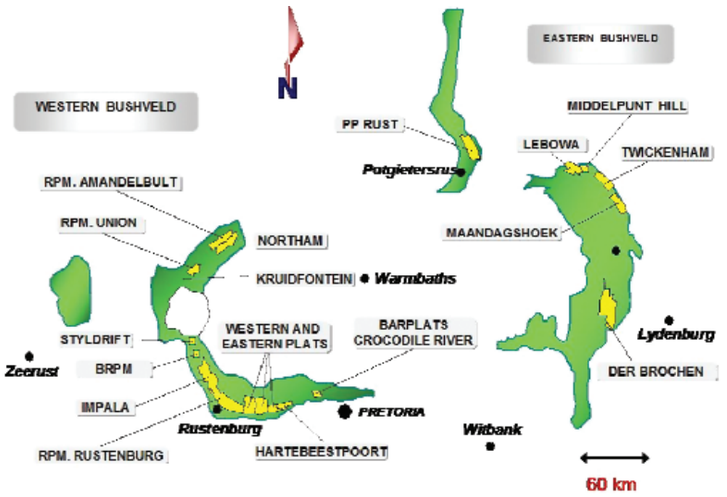

Figure 1—The extent of the Bushveld platinum exposure

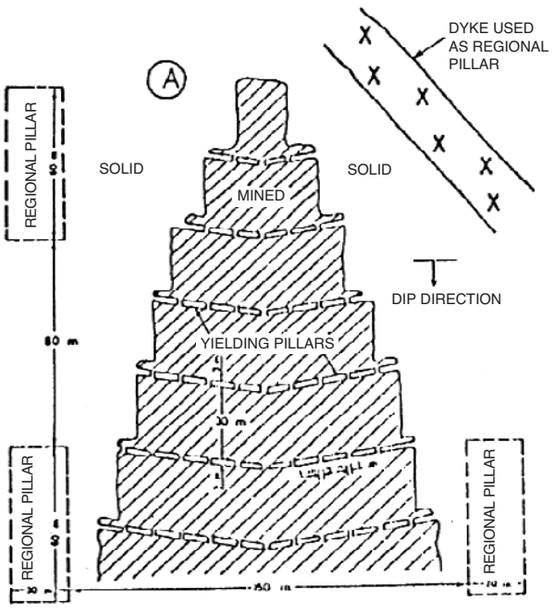

Figure 2—Plan view of a typical stope on one of the planar platinum orebodies

Instrumentation

Site description

An instrumentation site was established in a mine on the Merensky Reef in the north-west of the Bushveld Complex at a depth of 600 m below surface. A breast mining configuration was employed with chain pillars spaced approximately 30 m apart skin-to-skin on dip (see Figure 3). Two and three dimensional stress measuring cells were installed above Pillars 1 and 2 in Panel 13-18W-1E, as shown in Figure 3. Convergence measured at a distance of 1 m up-dip of the pillar edges was used to calculate pillar strain. Less accurate convergence measurements were also made about 3 m from the up-dip and down-dip edges of the pillars to observe the possible effects of foundation failure.

The size of the pillars at the site was about 3 m x 4 m in the dip and strike dimensions respectively and the stoping width was about 1.1 m. Pillars were cut at the edge of the advanced strike gullies (ASGs), immediately below the ASG, and the high side of the pillars were inclined at between 70° and 80° to prevent fractured slabs from rotating into the ASG (see Figure 4). The immediate footwall of the Merensky Reef at the site is a brittle anorthosite, which has a significantly higher uniaxial compressive strength than the reef. This anorthosite was exposed by the ASGs on the up-dip sides of all the in-panel pillars at the site.

The 13-16W-1E panel had been mined to its planned mining limit prior to the instrumentation installation. However, the lower panel was significantly behind the 13-16W-1E panel. The instruments were installed just behind and ahead of the advancing lower face. During the measurements the stope face continued to be mined and was stopped at the predetermined limit.

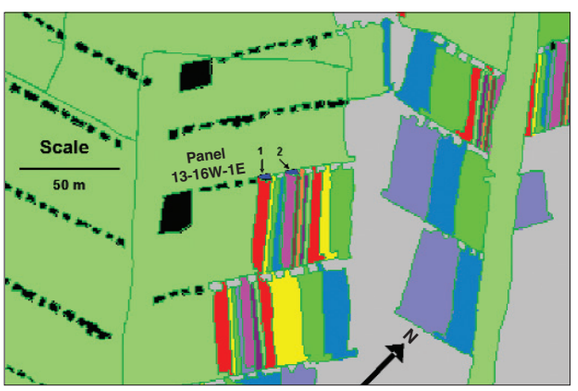

Figure 3—Stope sheet showing the instrumentation site

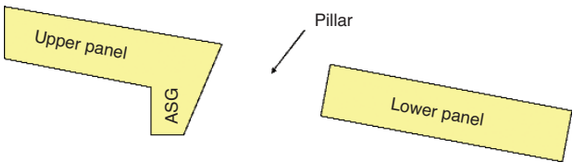

Figure 4—Diagram showing a section through a 'crush' pillar at the instrumentation site (not to scale)

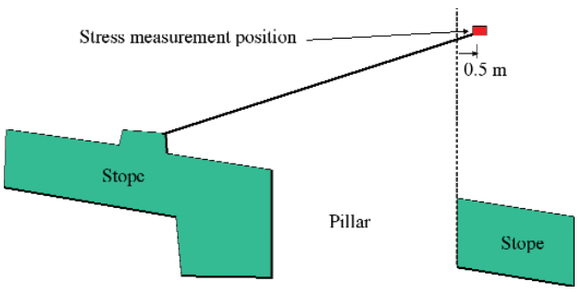

Figure 5—Section showing the instrumentation position above Pillar 1

Instrument positions

The high peak stress of the pillars meant that the measurements had to be made above the panel, away from the pillar edge, or higher than 5.5 m above the pillar centre. Stress measurements were made at one point above Pillar 1 and two points above Pillar 2. In order to relate these measurements to average pillar stresses, it was firstly 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 through an iterative process, using the Boussinesq analytical solution[1] and 3D MinSim[2] numerical models.

In the case of Pillar 2, the measured behaviour (at 5.5 m and 6 m above the pillar) could be simulated only if it was assumed that one of the adjacent pillars did not fail. The possible effects of vertical tectonic stresses on the field measurements were not considered, which appears reasonable for Pillar 1 but may be less appropriate for Pillar 2. A section and plan view showing the orientation of the instruments installed over Pillar 1 is shown in Figure 5 and Figure 6, respectively.

Two stress change cells were installed above Pillar 2 as shown in Figure 7 and Figure 8. A 3D CSIRO cell was mounted at a height of 6.2 m in the position shown as 1 in Figure 7 and Figure 8. Installation took place 2.5 m ahead of the advancing face, i.e. prior to pillar formation (see Figure 8). There is some doubt about the quality of information from this instrument as some of the gauges were unstable. A 2D doorstopper was mounted at 5.5 m above the inside edge of the pillar, as shown by position 2 in Figure 8 and Figure 9. The face position was almost in line with the cell at installation, as shown by the dotted line in Figure 8. A 2D field stress measurement was performed just behind the final cell position and this result was used in the back analysis of the Pillar 2 average pillar stress (APS).

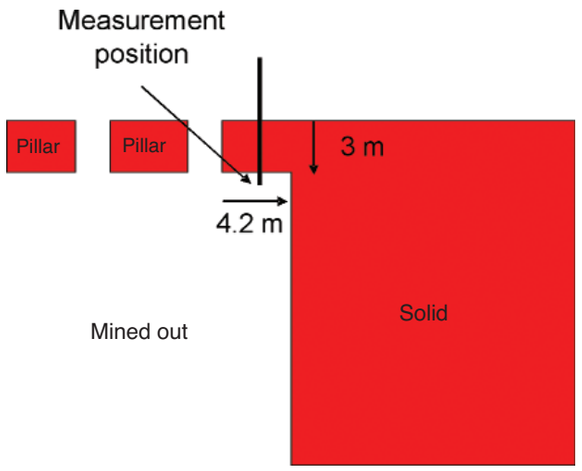

Figure 6—Plan view of Pillar 1 at cell installation

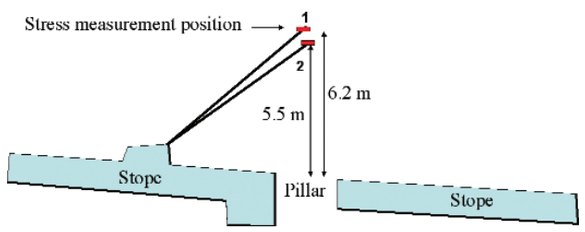

Figure 7—Section showing the instrumentation position above Pillar 2

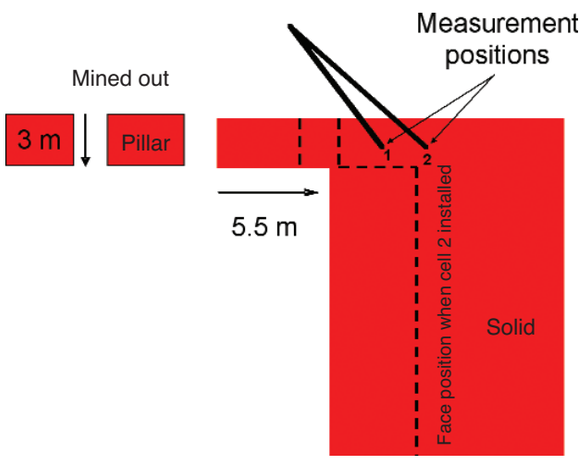

Figure 8—Plan view of Pillar 2 cell installations

A profile of residual stress measurements was obtained at regular intervals across the top of the pillars after the panel below had reached its limit and had stopped mining. The measurements were conducted from a borehole drilled at 5° above horizontal, or 23° above the strata, as shown in Figure 9.

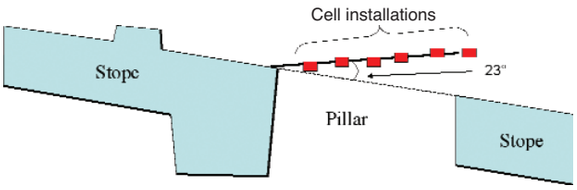

Figure 9—Section showing the measurement positions over Pillars 1 and 2 after pillar failure

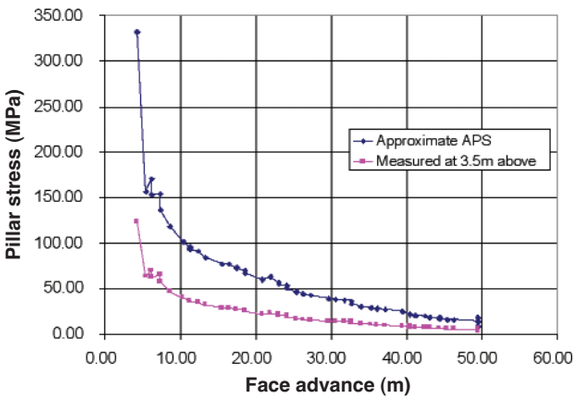

Figure 10—Pillar 1 stress results as a function of face advance

Results

The stress conditions recorded over Pillar 1 as well as the interpreted APS profile are provided as a function of face advance in Figure 10. Measurements initiated when the face had advanced 4 m, i.e. a 4 m long pillar stub had emerged from the face. The measurements suggest that pillar failure occurred when or before the instruments were installed. However, core-disking occurred while drilling the instrument borehole over the pillar (see Figure 11) suggesting that failure probably occurred during the blast, just after the instrument was installed. Thus the peak stress was recorded.

The range of stress error on the measurement point at 3.5 m above the pillar is shown by the dotted lines in Figure 12. The curve indicates that, in real terms, there is high confidence in the residual strength results at about 14 MPa, and that the peak strength is probably between 250 MPa and 370 MPa. It should be noted, however, that the measurements suggested a stress drop to 8 MPa after the face reached its stopping position. This was subsequently followed by a rise to 18 MPa, measured about three months later.

The shape and size of Pillar 1 were determined through off-set measurements. The average dimensions were 2.5 m wide by 5.5 m long, with an average stoping width of 1.1 m on the down-dip side. The up-dip side of the pillar was 2.2 m high.

The effects of adjacent pillars on the measurement positions above Pillar 2, at heights of 5.5 m and 6.2 m, were significant. Errors of up to 43 per cent are possible if the adjacent pillar reached double the stress of the monitored pillar. However, the errors associated with the possible stress profiles across the pillar were much less than for Pillar 1, and were estimated to be about 6 per cent.

The stress-face-advance curve for Pillar 2 is shown together with the instrumentation measurements in Figure 13. A peak stress of 374 MPa was determined assuming rigid pillar conditions. This means that the face effects and the peak pillar stress as determined by the numerical model, may have been under and overestimated respectively. In addition, the stress profile due to crushing at the pillar edges could have resulted in a further overestimate of the stress conditions by 6 per cent, as described above. Thus the peak stress for Pillar 2 probably ranged between 350 MPa and 374 MPa, which is slightly higher than for Pillar 1. The stress drop shown by Pillar 2 was more rapid than for Pillar 1, but the residual stresses were similar.

The average width of Pillar 2 was 3 m, which is slightly wider than Pillar 1. However, the length was 4.3 m, which was less than Pillar 1. The heights of the two pillars were similar.

Figure 11—Core-disking in the stress cell borehole above Pillar 1

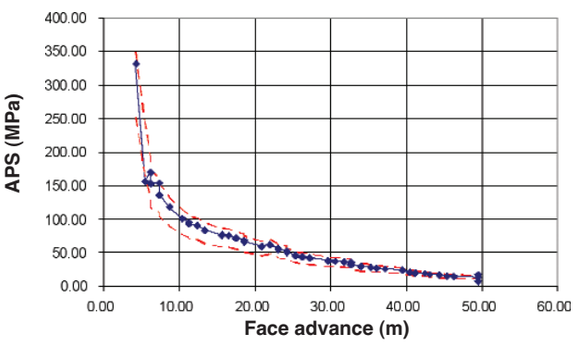

Figure 12—Pillar 1 stress results showing the possible error due to the range of likely stress profiles across the pillar

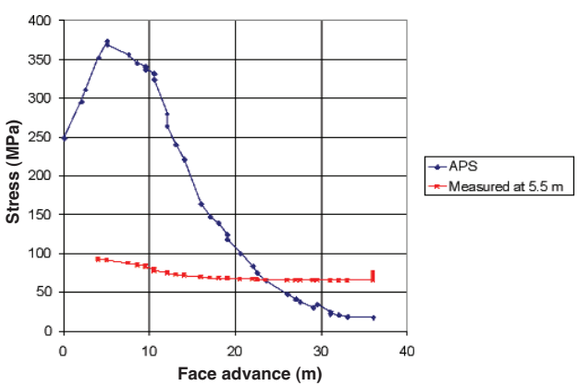

Figure 13—Pillar 2 stress results as a function of face advance

Residual stress profiles were measured using field stress instruments immediately above the pillars as described earlier. Most of these measurements were conducted using 2D doorstopper installations. The results reflect stresses in the plane of the long axis of the pillar in the sub-vertical and horizontal directions (see Figure 14). Only the sub-vertical stresses are shown in Figure 15.

The pillar sidewalls were highly fractured, as shown in Figure 16, indicating that the stress should be low here. This was shown by both pillars on the up-dip side (0 m in Figure 15). The down-dip measurements were performed about 1 m above the pillar and should therefore indicate stresses across a greater portion of the pillar width. Thus the non-zero measurements on this side of the pillar are reasonable.

The centre region of the pillar is confined and therefore able to carry a higher stress than the edges. The peak stress is therefore expected to be off-centre towards the shorter side of the pillar, on the down-dip side. This was shown by Pillar 2 but not by Pillar 1. The average pillar stresses, shown by the dotted lines in Figure 15, were calculated assuming the measured profiles are persistent along the length of the pillars. This assumption is probably incorrect and a better estimate is provided by a measurement made 3.5 m above Pillar 1. This measurement showed the average pillar stress to be about 3 MPa higher than the average calculated for Pillar 1 and provided in Figure 15. The final calculated residual stress on Pillar 2 was 18 MPa. This is 1 MPa greater than the average of the stress profile measured across the pillar after the face reached its limit position. Corrections for the heights of measurements above the pillars were not made. Thus the average results and some of the measurements are likely to be slightly higher than those shown in Figure 15.

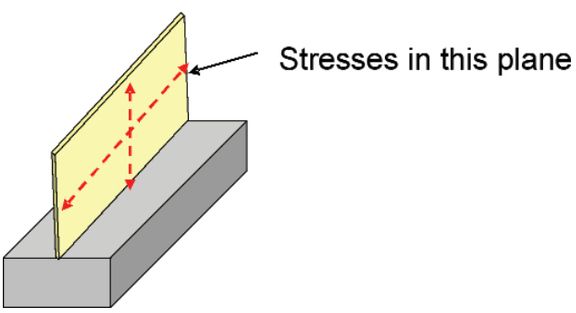

Figure 14—Sketch of a pillar showing the plane in which the 2D residual stress measurements were made

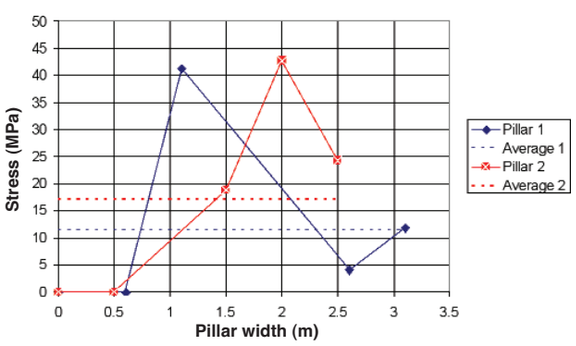

Figure 15—Stress profiles across the pillars measured after the down-dip face reached its stopping position. (Not corrected for measurement height.)

Numerical modelling

Loading and failure of the pillar was associated with ongoing mining as well as with time effects such as creep. At this stage, no distinction has been made between these two possible causes of loading and failure. Specimens of the Merensky Reef were previously subjected to triaxial testing and these test results have been used to calibrate numerical modelling parameters. A Mohr-Coulomb constitutive model, in which the parameters are variable with respect to the plastic strain, has been used. A two-dimensional, plane strain model of a section through the pillar has been simulated using FLAC3. Both vertical boundaries are located at mid-span on both sides of the pillar and allow unrestricted vertical displacement and no horizontal displacement. A hydrostatic pressure of 10 MPa was used to initiate the field stresses, after which the pillar was cut. Further loading, representing the three dimensional geometric effects of mining around this pillar, was achieved by forcing the top horizontal boundary downwards at a controlled displacement rate. The bottom horizontal boundary was not allowed to move in a vertical sense. The stress–strain curve for Pillar 1 (Figure 17) was used to calibrate the model.

Figure 16—Photograph showing fracturing on the ASG side of Pillar 2

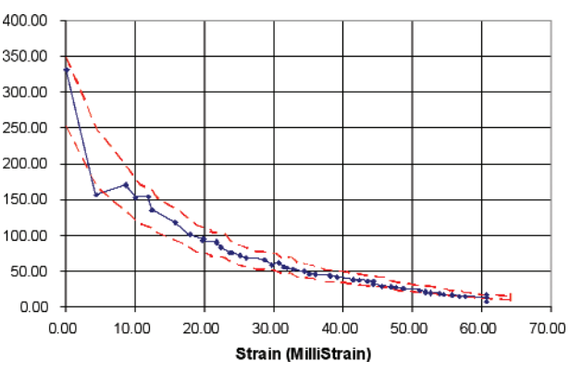

Figure 17—Pillar 1 stress–strain results showing the possible error due to the range of likely stress profiles across the pillar

Initial results (model M1) showed a maximum average pillar stress of around 180 MPa, after which increasing vertical deformation led to progressive failure. Average pillar stresses decreased rapidly to a residual value of around 5 MPa at a relatively low strain of 0.03. The measured strain at this residual level is approximately 0.06.

In order to match the numerical modelling results with the underground data, the modelling parameters that were originally calibrated against the laboratory data were adjusted. It was found that the average pillar stress was very sensitive to manipulation of the strength parameters at high plastic strain values, which effectively represents the residual portion of the pillar response. The cohesive strength ultimately drops to zero and the residual pillar strength is ultimately controlled by frictional resistance. However, both the peak as well as the residual strength of the pillar are extremely sensitive to the inelastic strain at which the cohesive strength reaches zero. Table I shows the parameters that have been used in the various models.

While it might be possible to obtain more realistic pillar strains by reducing the peak values and increasing the values for the strains, it was decided to minimize the data manipulations in this rather simple 2D model. The focus here is on the peak and residual values of the average pillar stress. The results of model M1 are provided in Figure 18. Peak and residual stresses of 180 MPa and 5 MPa were obtained respectively. These values are much lower than the observed values, as shown in Figure 17. After an iterative calibration process, strength parameters that provided a reasonable match were selected for model M2. The results of the M2 model show an average pillar strength of 320 MPa and an average residual strength of around 15 MPa (see Figure 18). The excessive brittle response is most likely associated with the simplified plane strain geometry that has been employed in this model.

Using the parameters from model M2, a possible change in pillar geometry has been investigated. In model M3, a siding has been created 2 m away from the pillar edge, while maintaining the original pillar width. This effectively resulted in a pillar with a larger width-to-height ratio, which caused an increase in peak as well as residual strength. This result is also displayed in Figure 18, while Figure 19 shows the vertical stress distribution, after pillar failure, in models M2 and M3. Maximum vertical stresses are located in the confined core of the pillar and it is clear that this core has shifted from the right-hand edge in model M2 towards the centre of the pillar in model M3.

Table I

FLAC model parameters

M1	Pl. ε	0	0.001	0.0015	0.002	0.003	0.05
	Co. (MPa)	50	22	16	11	4	0
	ϕ	10	40	43	45	47	45
M2 and M3	Pl. ε	0	0.001	0.002	0.004	0.01	0.02
	Co. (MPa)	50	47.5	45	40	25	0
	ϕ	10	30	40	45	45	42

(Co.=cohesion; ϕ = friction angle in degrees; Pl. ε is plastic strain)

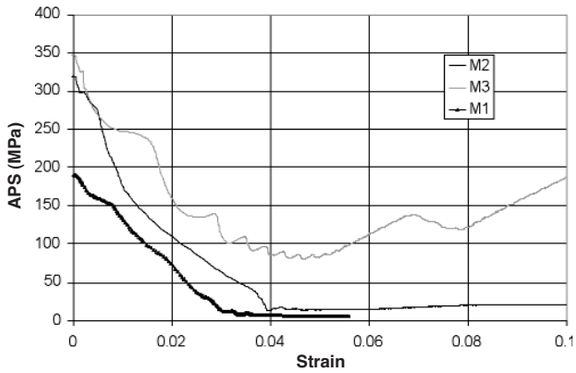

Figure 18—Stress strain results of the FLAC models

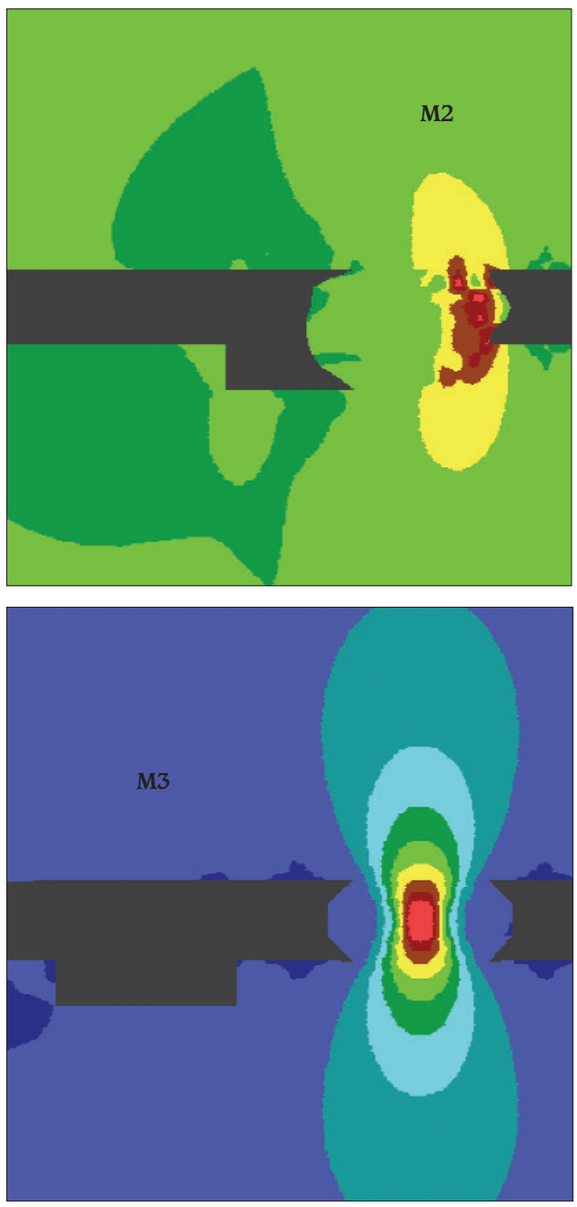

Figure 19—Residual stress distribution in models M2 and M3

The off centre location of the core in model M2 can be explained on the basis of the variation in confinement that is provided on both sides of the pillar. The gully creates a longer pillar sidewall, which enables failure to progress more easily into the pillar. Penetration of failure is relatively limited on the down-dip side of the pillar where the pillar wall is smaller.

The cutting of a siding away from the pillar edge particularly seems to affect the residual strength, which increased from 15 MPa to approximately 80 MPa. The peak strength increased from 320 MPa to around 350 MPa.

Discussion

Both the instrumented pillars failed before being fully cut, at 4 m to 5 m behind the face, as shown in Figure 20.

Three pillar bursts were investigated on the same mine where the stress measurements were undertaken. All the pillar bursts occurred on the first or second pillar back from the down-dip face, as shown in Figure 21. One of these bursts created a magnitude 1.2 seismic event, resulting in violent ejection of rock, as shown in Figure 22. This pillar was located 10 m to 14 m behind the 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.

MinSim numerical modelling suggested peak stresses of between 250 MPa and 350 MPa on the burst pillars. The instrumented pillars had similar peak strengths, suggesting a risk of bursting should the pillars not fail near the face as required. The risk could probably be reduced by using smaller pillars and hence lowering the peak load-bearing capacity. Of concern would be the consequent lowering of the residual strength and the ability of these pillars to support the hangingwall rock up to a distinct parting plane some 15 m to 20 m above the workings.

Back analyses performed by Roberts *et al.*[4] at Northam Platinum mine and Randfontein Estates Gold mine suggested that about 1 MPa support resistance was sufficient to arrest a backbreak. Partings that had opened in the hangingwall some 28 m above the workings at Northam Platinum mine were stabilized by backfill once a stress of about 1 MPa had developed in the backfill. A support resistance of 0.6 MPa is sufficient to carry a hangingwall height of 15 m to 20 m. If an extraction ratio of 92 per cent is assumed, pillar residual strengths of between 8 MPa and 13 MPa are required. The stress requirements are only slightly lower than the residual stresses provided by the measured pillars.

The first crush pillars on the Bushveld Platinum mines were introduced to Union Section by Korf[5] in 1978. The dimensions of these pillars were 1.5 m x 3 m with a height of about 1 m. The original pillars were thus significantly smaller than the pillars at the instrumentation site. Before the introduction of crush pillars, stopes at Union Section were supported on mat-packs. Serious problems 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, causing parting of the rock at the bottom contact of the Bastard Reef some 20 m above the stopes. At least three to four stopes were collapsing per month. The introduction of the pillars stopped the stope collapses in the mining area where they were introduced.

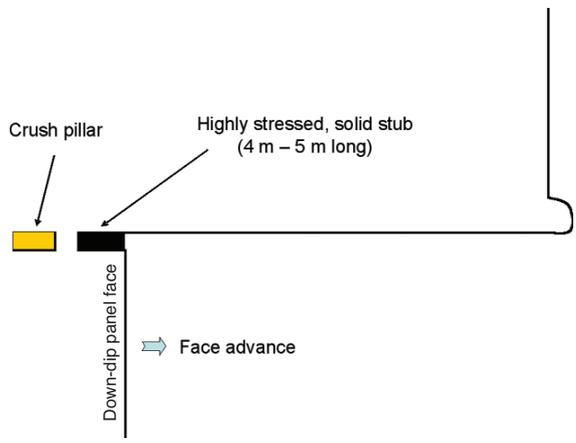

Figure 20—Sketch showing the face position where Pillar 1 and Pillar 2 failed

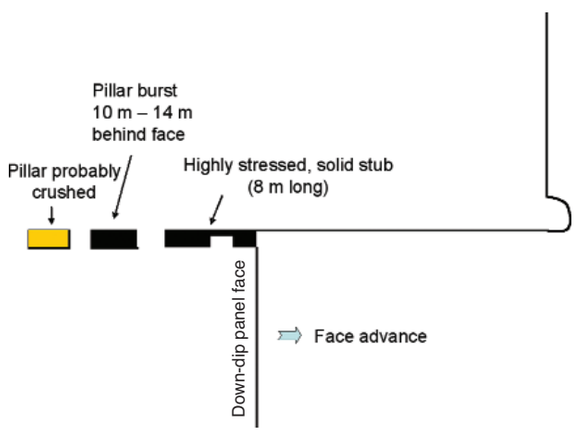

Figure 21—Sketch showing the face position where pillar burst occurred

Figure 22—Panoramic view showing the up-dip end of a burst pillar. Note the gully is full of rock fragments from the pillar burst

This area extended from 100 m to 700 m below surface and about 1 300 m on strike. The residual stresses of these pillars were never measured; however, the successful elimination of the backbreak problem suggests that the original pillars had a residual strength of at least 8 MPa. The FLAC modelling suggests a sensitive relationship between effective pillar width and residual strength, indicating that the pillars at the instrumentation site (with much wider dimensions) should have had significantly higher residual strengths than the Union Section pillars. The relatively low residual strength actually measured (for the pillar width) could not be attributed to jointing as the peak stresses were high. However, the pillars at Union Section were cut about 2 m down-dip of the ASG and therefore had equal heights on both sides of the pillar. It appears that the ASG has a detrimental effect on the residual strength if sidings are not left between the pillars and the ASG. Pillars could therefore be cut smaller if a siding were left between the pillars and the ASG.

Conclusions

The results of the investigation suggest that the pillars at the instrumentation site could be cut to smaller widths, thus increasing extraction and reducing the risk of pillar bursting. However, the current pillar shapes, affected by the ASG, appear to have a negative consequence on residual strength. The effects of the original crush pillars, introduced at Union Section, suggest that a sufficiently high residual stress can be achieved by 1.5 m wide pillars in a stoping width of about 1 m, provided 2 m sidings are cut between the pillars and the ASG.

References

1. Poulos, H.G. and Davis, E.H. *Elastic solutions for Soil and Rock Mechanics*. J Wiley & Sons. 1974.
2. COMRO. *MINSIM-D User's Manual*, Chamber of Mines of South Africa, Johannesburg. 1981.
3. Itasca consulting group, inc. Fast Lagrangian Analysis of Continua (FLAC), Vers. 3.2. Minneapolis Minnesota USA. 1993.
4. Roberts, D.P., Roberts, M.K.C., Jager, A.J. and Coetzer, S. The determination of the residual strength of hard rock crush pillars with a width to height ratio of 2:1. *J. South African Inst. Min. and Met*. vol. 105, 2005. pp. 401–408.
5. Korf, C.W. *Stick and pillar support on Union Section*, Rustenburg Platinum Mines. Association of Mine Managers of South Africa. 1978. pp. 71–82. ◆