

Safety in Mines Research Advisory Committee

Final Project Report

**Influence of regional support systems
(pillars and backfill) on local areas and
internal support requirements adjacent
to that regional support**

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Executive summary

The general influence of regional support on local stope conditions is not always well understood and a number of anomalous conditions, both positive and negative, have been reported in the vicinity of regional supports. Mining in similar rock conditions has apparently resulted in very different conditions, depending on the type of regional support used in the region.

An incomplete understanding exists of the effect of regional supports on the local conditions around them and the criteria that should be adopted for the safe design of local support in the vicinity of the regional support. There are many examples in the industry of anomalous conditions in the vicinity of regional support, sometimes good and sometimes adverse, and the reasons for such conditions need to be fully investigated and understood.

The topic of local support requirements adjacent to regional support is, therefore, investigated with the intention of providing recommendations and best-practice design guidelines for this potentially problematic zone in gold and platinum stopes.

Extensive ground motion and closure monitoring, studies of hangingwall conditions and numerical modelling analyses were carried out for a selection of different regional support (i.e. pillar and backfill) layout and reef horizon combinations. Although evidence arose from this work of differences in conditions possibly requiring different support design, it was felt that no absolute values could be provided for relevant design parameters. Rather, it was decided that a modified support design methodology be proposed that accounts for the additional influencing factors pertaining to the zone adjacent to regional support.

The primary conclusions of this work are:

- The issue of backfill influence on hangingwall conditions in the Vaal River mining region could not be adequately resolved. No direct evidence was observed of backfill creating worse hangingwall conditions; instead it was observed that poor backfill placement was associated with the less favourable hangingwall conditions.
- Generally, well placed backfill improves conditions in face areas if it is kept close to the face and conventionally designed working area support that fits in well with the backfilling/mining cycle is implemented. Conversely, quality is not assured if backfill is not well placed. Also large fill-to-face distances are the result of regular filling not taking place and these distances, together with inadequate working area support, lead to deteriorating and unsafe conditions. Combinations of the above poor practices are implicit in the findings of Squelch and Görtunca (1991) who state that accident rates are higher in backfill stopes than in conventionally supported stopes if less than 60 per cent of the stope is backfilled. To achieve the benefits of backfill, strict adherence to a well-established set of standards for both backfilling and working area support must be ensured.
- Dynamic closure, resulting from events generating PPVs, is reduced in the vicinity of regional support (pillars and well consolidated backfill).
- Support units in the vicinity of strike stabilising pillars will be required to withstand less dynamic and quasi-static closure than units in panels further away from the pillars.

- A greater relative increase in peak particle velocities (PPVs) will be encountered in the vicinity of strike pillars than in areas closer to the middle of the stope.
- Areas close to strike pillars, particularly gullies, have to sustain the cumulative effects of nearby events emanating from the pillar for the lifetime of the gullies.
- Conditions in gullies adjacent to backfilled panels are generally considered to be at least as good as if not better than those where conventional support is used, particularly under rockburst conditions. The question of whether to backfill up to the gully edge with or without internal reinforcing is currently being researched and did not form part of this project.
- The PPVs recorded close to the face adjacent to the recently placed backfill are higher in all monitored panels than the PPVs recorded further back in the areas where the backfill is already consolidated and provides a better support. Similar behaviour was found for all backfill sites at Kopanang, Tau Tona and Deelkraal mines. Any differences from site to site can be explained by variations in the local conditions.
- It is not possible to provide a generic design criterion for local area support in the vicinity of regional support, because of the variable nature of conditions that exist and the limited nature of the research study. It is, however, relevant to apply a methodology to the process of determining the criterion for each situation.

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1 Introduction

The primary output of GAP615 is:

- Best practice for design of support methods in the zone of influence of regional support structures for both pillars and backfill.

The secondary outputs are:

- Improved criteria and guidelines for the design of mine layouts at depth so as to reduce rockfall and rockburst damage and enhance worker safety by the determination of the effects of regional support systems (strike or dip stabilising pillars with or without backfill and backfill itself) on local areas (face and gullies) and the support requirements in those critical areas.

The envisaged impact of the project will be a reduction of rockburst damage and rockfalls in the zone of influence of regional support systems

1.1 Problem statement

The general influence of regional support on local stope conditions is not always well understood and a number of anomalous conditions, both positive and negative, have been reported in the vicinity of regional supports. Mining in similar rock conditions has apparently resulted in very different conditions, depending on the type of regional support used in the region.

An incomplete understanding exists of the effect of regional supports on the local conditions around them and the criteria that should be adopted for the safe design of local support in the vicinity of the regional support. There are many examples in the industry of anomalous conditions in the vicinity of regional support, sometimes good and sometimes adverse, and the reasons for such conditions need to be fully investigated and understood.

The benefits of establishing the appropriate local support are twofold. Firstly, the appropriate support will mean that such support is adequate to cope with the expected conditions that will prevail near or in the regions where regional supports exist, thus adding to the safety of the working places. Secondly local support in the vicinity of regional support systems will not be over-designed, thus reducing support costs considerably. An additional benefit of the work will be a better understanding of the influence of regional support systems. Previous work has sought to quantify the regional benefits of backfill and stabilising pillars and this research will build upon that base. Large regional areas of backfill now exist and an opportunity to assess the regional and local benefit of backfill is now possible. The interest in the use of dip pillars in ultra-deep mine layouts has concentrated on the regional support benefits but investigations need to be conducted into the effect of these regional supports on local support conditions and the best type of local support for such conditions.

Although a number of researchers have investigated various aspects of the problem, none appear to have done so comprehensively. In most cases the design of support for areas adjacent to regional support is done on an empirical basis, decreasing the spacing of support units until a satisfactory system is established. Some instability problems have been reported from these areas. A few rock engineers have reported that they have changed their method of support design to one based on a rating system.

1.2 Scope of work

This project is concerned with studying the effects of regional support on local areas for a range of typical regional support mining layouts used in gold mines. These include various arrangements of strike stabilising pillars and dip stabilising pillars on different reef horizons, e.g. Carbon Leader, VCR and Vaal Reef.

The project methodology is as follows:

- evaluation of current knowledge on the effects of regional support systems on local areas by means of a literature review;
- evaluate local support systems currently used adjacent to regional supports;
- numerical modelling to determine the effects of regional support systems on local areas;
- site investigations to determine the effectiveness of various local supports in the presence of regional supports;
- determination of internal support requirements;
- recommendations for appropriate local support with different regional supports;
- final report; and
- technology transfer.

1.3 Report structure

- Section 1: Introduction and background
- Section 2: Literature review
- Section 3: Research methodology and experimental approach
- Section 4: Results of underground site investigations
- Section 5: Discussion of findings
- Section 6: Technology transfer
- Section 7: Conclusions and recommendations

2 Literature review

This literature survey examines the previous studies related to regional support systems in order to determine:

- the usefulness and relevance of previous studies to the current project;
- the relevance of major findings to the current project;
- the nature of anomalous rock mass behaviour adjacent to regional support systems and the causes of this behaviour;
- the gaps between the current knowledge and the knowledge required for the completion of the project; and
- a focused direction for future studies.

2.1 Stope and gully support

Roberts (1995) conducted a study on stope and gully support, the primary objectives of which were the development of a rationale for the design of stope support systems and design criteria for the support of stope working areas and gullies. The major outputs were the following:

- determination of support resistance criteria for rockfalls and energy absorption criteria for rockbursts for the VCR, Carbon Leader and Vaal Reef;
- development of a stope design methodology where the stope face and back area are evaluated separately;
- development of a numerical model of stope support-hangingwall interaction;
- recommendations on desirable force-deformation curves for gully packs;
- compilation of a stope support catalogue of commonly used support units in Witwatersrand mines; and
- determination of support resistance criteria and energy absorption criteria for gully support.

The separation of the support design methodology into face, back area and gully area is important, as the requirements of each area are different.

2.1.1 Improved support design by an increased understanding of rock mass behaviour around the Ventersdorp Contact Reef

The primary objectives of this project conducted by Roberts and Guler *et al.* (1996) were the development of a conceptual model for rock mass behaviour around VCR stopes and the design of efficient support systems for static and dynamic conditions. Major outputs include the following:

- collation and review of all past rock engineering and geological work on the VCR;
- definition of geotechnical areas for the VCR mining areas;
- identification of fracture patterns in different geotechnical areas;
- evaluation of rock property data for the immediate footwall and hangingwall rock types;
- assessment of variability in fracture patterns and stope closure between geotechnical areas;
- analysis of relative seismic intensity with respect to geotechnical variations;
- numerical analyses to quantitatively understand rock mass behaviour;
- analysis of data from fatality database; and
- identification of a suitable approach for support design in VCR stopes.

A methodology to define geotechnical areas was developed and six primary geotechnical areas were delineated for the VCR. Potential exists, however, for further subdivision of the primary geotechnical areas. The concept of geotechnical areas was shown to be valid and a useful basis for support design.

Distinct fracture patterns were found to occur in different geotechnical areas associated with different combinations of footwall and hangingwall rock types. Large differences were found to exist in the properties of the hangingwall and footwall strata, especially between the 'hard' Alberton Porphyry Formation and the 'soft' Westonia Formation lavas. Differences in rock mass behaviour were found to exist between the hard and soft lavas.

2.1.2 Stope face support systems

Daehnke *et al.* (1998) completed a multi-disciplinary project on stope face support systems. The primary output of the project was the determination of geotechnical areas across the gold and platinum mines to form the basis for future understanding of the rock mass behaviour around reefs where the hangingwall and footwall rock types differ, leading to improved site specific support systems for both static and dynamic loading.

Geotechnical areas of eight Witwatersrand and two Bushveld orebodies were delineated on the basis of footwall and hangingwall characteristics. An indication was provided on the influence of certain rock engineering aspects on local and regional support. These aspects include stope stability, fall of ground characteristics, attitude and frequency of mining-induced fracturing, seismicity and punching of pillar support and pillar strength.

Anticipated rock mass responses for various geotechnical areas are presented in Table 2.1.1.

Table 2.1.1 Anticipated rock mass response as predicted for the defined geotechnical environments

	Footwall soft ¹	Hangingwall Soft ²	Competent FW/HW assemblage ³	Incompetent FW/HW assemblage ⁴
Merensky Reef	tbd	tbd	tbd	tbd
UG2	tbd	tbd	tbd	tbd
Ventersdorp Contact Reef (hard lava)	(X)	n/a	X	n/a
Ventersdorp Contact Reef (soft lava)	(X)	X	(X)	(X)
VS5	X	n/a	(X)	(X)
Witpan (8a)	X	X	n/a	X
Big Pebble Marker	X	X	n/a	X
B-Reef	XX	X	n/a	X
Leader Reef	X	X	n/a	X
Vaal Reef	(X)	(X)	(X)	minor
Carbon Leader	with increasing depth	n/a	X	n/a

tbd = to be determined

n/a = not applicable

(X) = occurs locally

1 = punching of support; pillar foundation failure; footwall bulging/ride

2 = punching of support; pillar stability

3 = relatively high frequency of mining induced fracturing; relative low closure rates; relatively severe face abutment stress environment (face bursting and seismicity)

4 = relatively low frequency of mining-induced fracturing; relatively high closure rates

Regional support performance addressed factors such as pillar strength and foundation failure or hangingwall punching. Local support performance, especially, considered the footwall and hangingwall punching of support; thereby also identifying areas where the application of headboards is desirable. Stope stability was predominantly predicted through the delineation of soft footwall regions, where footwall bulging may occur, and the occurrence of major hangingwall partings that also control fall of ground characteristics, such as their heights. The delineated competency of the footwall and hangingwall assemblages provides an indication with regard to closure rate and the attitude and frequency of mining-induced fracturing.

A site-specific support design methodology was formulated on the basis of tributary area concepts and stable hangingwall spans. The methodology is suited to mines in the intermediate to deep depth range.

Closure-ride results are available for increasing distances away from a strike pillar. Other measurements were taken a panel below the pillar and can be used for comparison to determine the extent of the influence of pillars on rock mass behaviour.

2.2 Stabilising Pillars

2.2.1 Deep mine layout design criteria

Vieira *et al.* (1998) conducted an in-depth study (GAP 223) on deep mine layout design criteria with one of the outputs covering stabilising pillars. Maccelari (1998) concentrated on the seismic aspects of the study. The performance of strike stabilising pillars impacts on stope face conditions and on stability in the back areas (long-term stability). The following observations were made:

- Stabilising pillars will generate seismic events of magnitude $M \geq 2$ at some stage in their lives, regardless of their width or associated dip span.
- Stabilising pillar width did not have an influence on the level of back area seismicity except, possibly, where leads and lags are kept small, as at Western Deep Levels.
- Influence of secondary geological structures within stabilising pillars was more localised on the Carbon Leader than on the VCR due to lower stoping widths and differences in strengths of the hangingwall and footwall lithologies of the two reefs.
- Dip spans did not have an influence on the local level of face area seismicity, probably because the longwalls and stabilising pillars act as a system rather than as isolated longwalls.
- Stabilising pillars exhibited a delayed response to changes in stress levels resulting from face advance.
- Regular shapes of stabilising pillars were associated with a lower seismic hazard than those with irregular shapes.
- It was necessary to determine the width at which changes in failure mode occurred and to use this as a guideline for a minimum pillar width.
- Depth and type of geotechnical area will influence the minimum required pillar width.
- Average pillar stress and ERR alone are inadequate design criteria for strike stabilising pillars.
- An improved understanding of the inelastic behaviour of stabilising pillars is necessary to enable more realistic modelling.
- Stronger foundation rock types experience higher levels of seismicity in terms of both number of events and magnitude.
- Differences in footwall and hangingwall lithologies may play a significant role in pillar behaviour.

2.2.2 Pillar failure

Hagan (1990) indicated that pillar foundation failure resulting in seismic activity and rockbursts has been experienced on some of the stabilising pillars at Western Deep Levels Gold Mine. A wedge-type failure mechanism was proposed where the solid pillar core punched into the failed foundation rocks resulting in footwall lifting on the down-dip side below the pillar.

Lenhardt and Hagan (1990) proposed four mechanisms for stabilising pillar failure. The simplest case is a crush-type mechanism associated with relatively narrow pillars. The actual pillar fails leaving the hangingwall and footwall relatively intact. Lenhardt (1989) found this type of failure to be easily identifiable from its seismic signature. It produces dilational first motions on recorded seismograms. These events were more common on the VCR, where stoping widths are higher than on the Carbon Leader, possibly influencing the width-to-height ratio. The other three mechanisms involve shear slip along pillar edges where the foundation fails and the pillar remains relatively intact. A shear type event is characterised by slip along one plane, below and parallel to the pillar edge. A compressional, double-couple first motion indicates a shear

mechanism. In the punch-type event, shear planes form on both sides of the pillar and extend into the footwall. The mechanism is considered more likely to be a simultaneous uplift of the footwall into stopes on either side of the pillar, leaving the pillar core relatively stationary. First motions of these events are more complex. Fracturing of the pillar core may occur after the main seismic event. A single instance has been recorded of simultaneous shear-type events occurring along the edges of two adjacent pillars in a deep VCR setting. First motions were also complex for these coupled events. The hangingwall of the stopes involved in shear-type failures was relatively unaffected, a situation attributed to higher competencies of the hangingwall rocks.

There is no current precise definition of stabilising pillar failure or stabilising pillar foundation failure (Maccelari, 1998). Variable rock mass strengths and inelastic behaviour of the rock mass complicate the possible definition of these terms.

Maccelari (1998) further discussed two conceptual categories of failure, being failure in a design sense and failure in a rock mechanics sense. 'Design failure' occurs when a stabilising pillar no longer provides adequate protection to working areas. 'Rock mechanics' failure occurs when the load applied to a pillar exceeds the elastic limit of the rock mass comprising the pillar or its foundation. Maccelari concluded that when a stabilising pillar is functioning from a 'design' point of view, it is highly likely that it may have a higher potential to fail from a 'rock mechanics' point of view. Reported failure of stabilising pillars may refer to failure in a design sense where a violent release of energy results in damage to stope faces or service excavations, and support provided by the pillar is substantially reduced for a period of time.

2.2.3 Pillar behaviour

Deliac and Gay (1984) noted that certain dykes influence the level of seismicity in the vicinity of stabilising pillars. Certain geological structures intersecting pillars may make them prone to rockbursts.

It was observed that most events occurred in the hangingwall mainly at between 50 and 150 m above the reef but as distant as 350 m away from the pillar. This was attributed to greater elastic moduli and compressive strengths of the hangingwall compared to the footwall, allowing the former to store more strain energy than the latter.

Brummer (1987) described a pillar deformation mechanism where the pillar is squashed vertically and squeezed out horizontally due to steep shear fracturing. It was found with the aid of boreholes that the pillars were dilated laterally over a height significantly more than the actual stoping width, i.e. within the rock mass above and below the stope horizon. Lenhardt (as quoted by Maccelari, 1995) conducted detailed observations at a failure site of a 35 m wide pillar. A seismic event had originated close to the face, but energy was transmitted up to 100 m behind the face with associated damage. Closure was higher on the up-dip side (215 mm) than on the down-dip side of the pillar. Ride components were found to indicate a 'flow' of rock mass from beneath the pillars into the stopes. Ride in the strike direction was also noted.

Diering (1987), Hagan (1987) and Turner (1987) have documented poor stope and gully hangingwall conditions, immediately up-dip of 20 m wide strike stabilising pillars at Western Deep Levels. Large collapses, to above the Greenbar occurred in gullies and sidings adjacent to the pillar. Rehabilitation of the 4 m to 5 m high brows was onerous, hazardous and not always successful. Diering attributed this behaviour to zones of failed rock defined by Griffith's failure criterion. Above the pillar, the zone of failed rock lay above the gully and any large seismic event would shake it down. In the down-dip area, the failed, crushed zone lay underneath the gully. Hagan noted that poor hangingwall conditions extended 10 m up-dip from

the pillar edge. Pillars 30 m wide on the VCR were in comparatively better condition. Minsim modelling revealed zones of failed rock in the hangingwall up-dip of the pillar and in the footwall down-dip of the pillar, similar to Diering's findings. This failure was in accordance with Hoek and Brown's failure criterion. Displacements consistent with Brummer's (1987) pillar deformation mechanism were measured for a 20 m pillar. An outward 'flow' of rock away from a highly stressed pillar was suggested by ride measurements. Modelling and in-situ studies indicated that the up-dip problem may be alleviated by increasing pillar widths, a recommendation that was put into practice in 1985. Diering (1987) reported that the change in pillar widths to 40 m was initially encouraging. Lenhardt (1990) reported on accident statistics but did not separate those accidents for 20 m pillars (pre – 1985) from those for 40 m pillars (post – 1985). Turner attributed the poor hangingwall conditions to closely spaced fractures sub-parallel to the hangingwall along with joints and face parallel fractures. This problem was less evident at ERPM, where cross-beds in the hangingwall prevented the formation of hangingwall parallel fractures. The prevention of hangingwall parallel fractures was seen as the most cost-effective way of alleviating the hangingwall instability problem. This could be achieved by reducing the extent of fracture formation by mining only the gully as an advance heading. Support tendons installed in the hangingwall could support the resultant narrow arch of fractures.

Hagan (1990) described phenomena related to 20 m wide strike stabilising pillars at Western Deep Levels. A hangingwall stability problem due to fracturing up-dip of the pillar resulted in an increase in rockfall related accidents. Pillars were sometimes found to deform in an unstable manner, especially in back areas.

Lenhardt (1990) mentioned the presence of mining induced fractures, which tend to concentrate along the edges of pillars and therefore decrease the competency of the overall rock mass associated with the present 40 m pillars. Accidents were found to be more serious on the deeper Carbon Leader horizon. This was attributed to low stoping widths together with extensive footwall uplift that normally accompanies pillar events. The majority of accidents were found to occur on the up-dip side of pillars through rockfalls associated with seismicity. Accidents on the down-dip side, however, were more severe because of footwall uplift, causing extensive closure of the stopes. Pillar-induced seismicity was found to be evenly spread throughout the day and affected follow-behind haulages, strike gullies and reef drives more than stope working areas. Pillar widths and spans between pillars were found to have no significant influence on seismicity. Geology was found to influence pillar stability significantly. Pillar stability was found to be greater in areas underlain by a shale footwall, possibly due to plastic deformation of the shale into the stope.

Significant closure occurring in a step-wise fashion accompanied major pillar seismic events (Lenhardt and Hagan, 1990). Initial pillar foundation failure occurred up to 100 m behind the stope face and recurred once a further 60–100 m of face advance had been accomplished. Shear-slip events were related to well-defined shear planes located at the pillar boundaries and parallel to the pillars. The mechanism involved a slip along one edge of the pillar into the footwall, accompanied by uplift of the footwall in stope gullies.

Footwall geology influences pillar performance to a certain extent. Lenhardt (1992) found that VCR stopes underlain by quartzitic footwall experience higher seismicity levels than areas with soft, shale footwall. The shales deform plastically and do not permit shear stresses to build up.

Yilmaz *et al.* (1993) conducted a back analysis of off-reef strata displacements within the proximity of stabilising pillars. They concluded that stabilising pillars resist some of the elastic movement of strata into stopes. The magnitude of the movement could not be directly inferred from the results since the methodology using survey pegs was not sufficiently precise.

One rockburst investigation (Durrheim, 1997) involved the collapse of the gully immediately above a 40 m wide stabilising gully. The hangingwall failure mechanism was attributed to down-buckling of the Greenbar, caused by up-dip dilation of the pillar during the event.

2.2.4 Fracturing in and around pillars

Fracturing in and around pillars is known to reduce their effectiveness and also plays a role in pillar instability and may indicate possible foundation failure. Analysis of the fracturing helps to identify inelastic deformation mechanisms occurring in and around pillars.

Brummer (1987) conducted a detailed study of fractures associated with 20 m wide stabilising pillars using stereophotography. An irregularly shaped pillar was completely fractured with shear displacements of up to 200 mm. At another site, mining induced fracturing extended 7 m into the pillar and a regular pattern of shear displacement was observed. Boreholes drilled into a pillar showed lateral displacements in response to 20 m of mining. These displacements occurred over a height of more than the stope width.

Hagan (1987) studied fracturing on the faces of ventilation slots in 20 m wide pillars. The entire pillar was fractured, with the most intense fracturing occurring along the down-dip side of the pillar (up to 26 per metre), whereas the up-dip side had a density of 18 per metre and the centre 6-8 per metre. Fractures in the central portion are parallel to the pillar edge and near vertical, whereas more variation is seen toward the pillar sides. Two phases of fracturing were recognised, corresponding to mining above and below the pillar. Fractures were found to swing away from face parallelism in the vicinity of pillars. Dips are generally shallower in these instances.

Ozbay and Ryder (1989) identified three fracture patterns from laboratory tests on norite and quartzite. The first type is extensile fracturing parallel to the edges of pillars. The second type is formed due to compression or shear and is near vertical, occurring near the edges of the pillar. The width of the fractures is greatest near the pillar and thins with distance from the pillar. The third type propagates from the second type towards the centreline of the stope.

Turner (1989) described fracturing in lead and lag situations. The overhand geometry studied may be likened to a strike stabilising pillar with a panel below it. Shallow hangingwall parallel fractures formed in the stope hangingwall below the pillar siding. Face parallel fractures curved and flattened as they approached the pillar siding. Steep, siding parallel fractures formed parallel to the edge of the siding.

Handley (1996) identified open shear fractures in the footwall, dipping vertically and oriented parallel to the pillar where foundation failure has occurred. These features may extend for up to 50m on strike.

Handley *et al.* (1997), while studying 40 m wide pillars at 2470 m below datum, found that fractures did not extend significantly more than 5m into a pillar and there were no fractures beyond 7 m. Ground penetrating radar indicated no significant increase in fracturing as mining advanced. Fracturing was oriented vertical or near vertical.

2.3 Backfill

Backfill is being placed in South African gold mines as part of their strategy to improve safety in underground workings, especially in the stope face area, by reducing the potential for the

occurrence of rockfalls (and therefore rockfall accidents), particularly in situations of deep-level mining and adverse geological conditions.

CSIR Miningtek (previously Chamber of Mines Research Organization (COMRO)) has extensively researched Backfill as a local and regional support. This includes the correct placement of backfill to maximise the regional benefits, placement properties of effective backfills, their *in situ* behaviour and the benefits of backfill placement on a local and regional scale.

The importance of backfill as a local support is highlighted by analyses of accident data from previous work by Grtunca and Squelch (1990), which has consistently shown that the majority of rockfall related accidents occur within 10 m of the stope face and in the access gullies. Work carried out by Squelch and Grtunca (1991) has shown that the use of backfill can bring about a reduction in the incidence of rockfall accidents in the stope face area, provided that the percentage of backfilling is greater than 60 per cent and the fill-to-face distances are kept to less than 6m. Moreover, rockburst damage was found to be less significant in backfilled panels than in conventional panels, when the distance between the backfill and the face did not exceed 6m and good face-area support was installed.

Reports by Adams *et al.* (1991), Grtunca and Adams (1991) and Grtunca *et al.* (1989) have described the behaviour of backfill at a fixed point in the fill, as well as the complete three-dimensional backfill rib behaviour. This work has given a model for understanding the behaviour of the rock surrounding backfilled stopes as a result of the stress build-up in the backfill and forms the basis for understanding the local behaviour of the rock surrounding a backfilled stope.

Underground observations by COMRO have also shown that backfill increases the stability of the hangingwall during seismic events, and Squelch (1990) has identified a compressive horizontal stress regime in the hangingwall of the backfilled panels, as opposed to unfilled panels that exhibit a component of tensile stress in one direction. The stresses acting parallel to the stope hangingwall of backfilled stopes were generally compressive while similar stresses in unfilled stopes were compressive perpendicular to the face, but tensile parallel to the face.

Adams *et al.* (1990) examined the behaviour of backfill as a local support under seismic conditions. This work showed that the backfill reduced the vibration time of the rock, following a seismic event and that overall ground motion was damped by backfill.

2.3.1 *In situ* behaviour of backfill as support medium

The placement of backfill or sandfill to reduce rockbursting was common in the deeper Witwatersrand mines during the 1920s and 1930s, since it was found that, with respect to rockbursts, faces in sand-filled stopes gave less trouble than those supported by ordinary methods (Watermeyer and Hoffenberg, 1932). More recently, Close and Klokov (1986) reported that stopes in backfilled areas on West Driefontein Mine that had been subjected to dynamic loading suffered little damage compared with unfilled stopes immediately below them, where the damage was extensive. Gay *et al.* (1988), in a survey of damage in filled and unfilled stopes caused by seismic events of magnitude 2.1 to 3.1, confirmed the observations of Close and Klokov that the damage in filled stopes was generally less than in unfilled stopes. They also noted that gully conditions were much better in backfilled stopes than in conventional stopes after rockbursts. However, despite these observations, they were unable to conclude that they had proved the greater efficiency of backfill as a local support than conventional support methods.

For six years COMRO monitored the *in situ* performance of various backfill materials. The main objectives of the monitoring were:

- (i) to quantify the *in situ* behaviour of various backfills used on the gold mines and to compare their performances with one another and also with conventional support systems such as timber packs;
- (ii) to provide input to the understanding of backfill as local support for reducing rockfalls; and
- (iii) to calibrate and develop constitutive models to simulate the *in situ* behaviour of backfill materials.

Metallurgical plant tailings and waste rock from the development of off-reef tunnels are the two main sources of backfill materials. There are four types of commonly used backfill material on South African gold mines: classified tailings, dewatered tailings, comminuted waste and cemented tailings (Jager *et al.*, 1987 and Gürtunca *et al.*, 1989). The stiffness (represented in terms of porosity) of these materials varies from one to another.

Of great importance, in the evaluation of the performance for the various types of backfill used, has been the measurement of the *in situ* behaviour of the backfill and its effect on the surrounding rock mass. The deformability or *in situ* modulus of the rock mass differs from the Young's modulus of intact rock, mainly owing to the existence of joints, faults, and stress-induced fractures within the rock mass. Different geological layers influence the *in situ* modulus of the rock mass in different ways.

2.3.2 Stress measurements in backfill

The stress-strain behaviour of backfill measured underground gives an indication of the behaviour of the rock mass surrounding a backfilled stope (Gürtunca and Gay, 1993). It was shown that cemented backfill was stiffer than uncemented backfill and that, as the porosity of the uncemented backfills increased, their stress-strain response became less stiff. The ultimate stiffness of uncemented backfill materials is important in the design of regional support based on backfilling. The smaller the strain at which the backfill attains the ultimate stiffness, the more effective it is as regional support.

Adams, Gürtunca and Squelch (1991) monitored the stresses at different locations in a backfill paddock and this information, together with measurements of rock mass behaviour, provided the information needed for the construction of a three-dimensional conceptual model as shown in Figure 2.3.1. Three types of fill material were considered for the investigation: namely, dewatered tailings, comminuted waste and classified tailings. The model provides an impression of the overall stress topography across a backfill rib at a specific time after placement. It has been useful for explaining the stress and closure distribution in backfill and the surrounding rock mass to production personnel at the gold mines, and has also provided valuable information for numerical modelling.

The model in Figure 2.3.1 shows that the stresses at the edge of the backfill are low, but not zero, as a result of the retaining force provided by geotextile bags, planks, or packs. The daily vertical stress profiles and equal strain vertical stress profiles were the two types of approaches used to present the results. The daily vertical stress profiles display the actual vertical stress distribution in a backfilled rib. The results from underground monitoring (Adams, Gürtunca and Squelch, 1991) revealed that the maximum stress occurred at a transition line, which is located between 1 m to 3 m from the backfill edge. The vertical stress for dewatered tailings, reaches 6-7 MPa at 14 per cent strain, for comminuted waste it reaches 3-4 MPa at 7-8 per cent strain, and in classified tailings 8-11 MPa at 20 per cent strain. The stiffness of the backfill increases

with time owing to the closure, and the transition point will simultaneously get closer to the backfill edge. The fill material is in confined compression between the transition points on opposite sides of the rib, and the generation of stress is governed primarily by the stoping width and the closure in that region assuming constant placement quality. The narrower the stoping width and the higher the closure rate, the greater will be the stress. The k-ratios (the ratio of horizontal stress to vertical stress) in the backfill bag are generally constant and vary between 0.3 and 0.6 for all three backfill types.

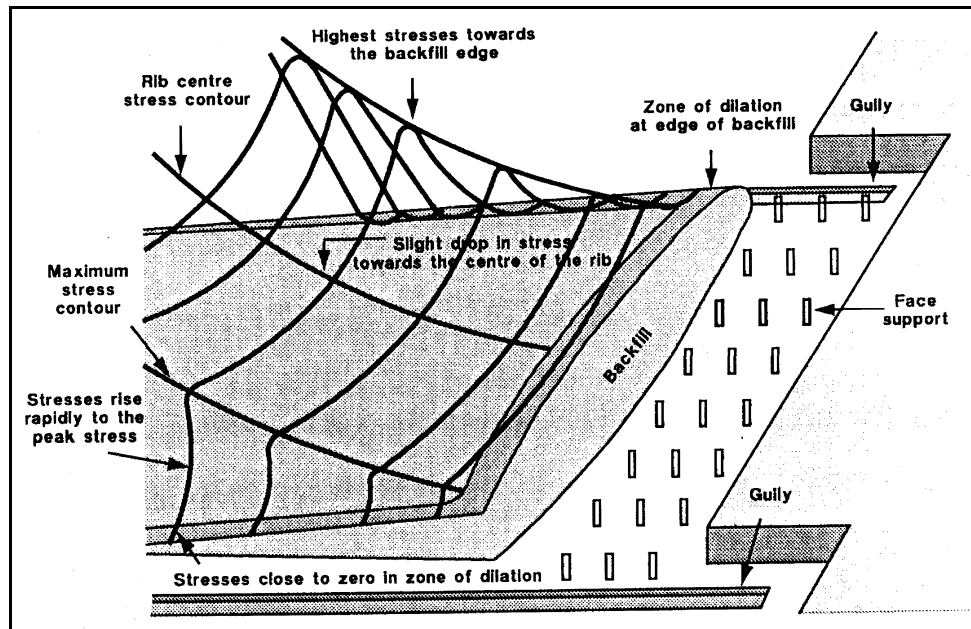


Figure 2.3.1 A schematic three-dimensional representation of the vertical stress topography of a backfill rib at a particular time (after Adams et al., 1991)

The range of stresses expected to be generated in *in situ* backfill is large. Where backfill is being placed for hangingwall control, the maximum stress likely to be encountered is probably less than 10 MPa. In areas where backfill is being used for regional support, stresses will be very much higher, approaching the pre-mining virgin stress. At depths of 4 to 5 km, this stress would exceed 100 MPa.

2.3.3 Closure measurements

Gürtunca and Adams (1991) also studied the variation of closure along the dip direction of a filled panel as the face advances. Three tri-axial stations (C1, C2 and C3) were located 3m, 5 m and 11 m away from the edge of the paddock respectively. In addition, a closure station was installed in the gully outside the backfill paddock. The curves shown in Figure 2.3.2 indicate the closures measured at each closure meter on the same day. The upper curve shows the measurements taken about 2 weeks after the installation of the meters, when the face was 8.5 m from the instruments. The lower curve which displays the maximum closures measured at each station was plotted from data obtained about 9 months after installation, when the face had advanced 34 m from the instruments.

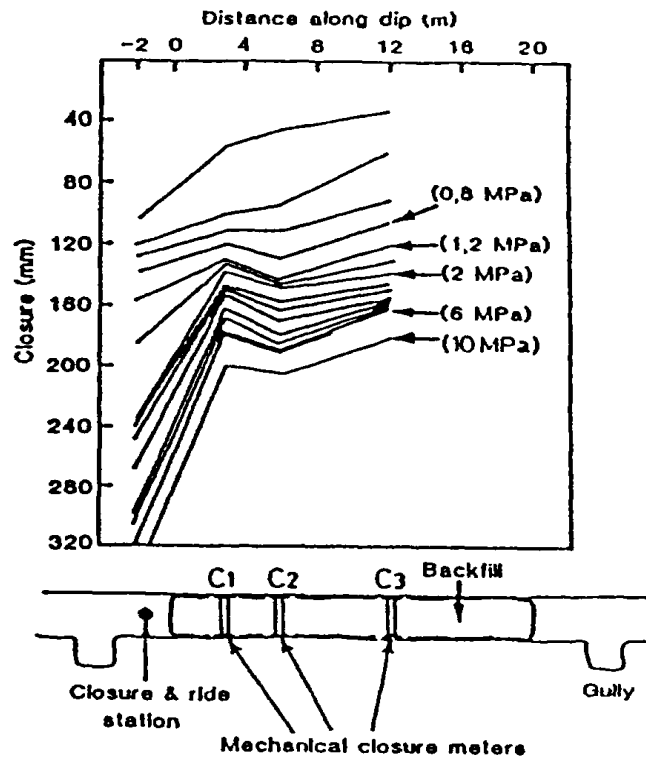


Figure 2.3.2. *The distribution of closure across a backfill paddock (after Gurtunca and Adams, 1991)*

Figure 2.3.2 clearly shows that the closure measured inside the fill is 40-50 per cent less than that measured at the closure station outside the fill adjacent to the gully. However, the initial closure at all stations develops more or less equally up to the point where the vertical stress in the backfill reaches about 1 MPa. The closure inside the panel is restricted while the station in the gully continues to be displaced at a relatively high rate until a stress of 3 MPa is developed in the fill. With higher backfill stresses, the amount of closure developing inside and outside the fill is fairly similar. This type of behaviour of the rock mass in response to the stress development in the fill (i.e. bed separations and dilations) may take place only when the stress in the backfill is less than about 3 MPa. At higher stresses, the strata become clamped, and the inelastic deformation is significantly reduced.

A consequence of the high initial differential movements between points inside and outside the fill is that rockfalls in the gullies are more likely to occur in the period before the rock mass is clamped and when the fractured hangingwall becomes subjected to tensile strains induced by differential movements. Closure measurements have been carried out along gullies and inside panels at the monitoring sites to monitor the influence of different types of backfill on closure by comparing results from filled and unfilled stopes. The results revealed that the closure per metre of face advance and per day are significantly less in filled panels than in unfilled panels, though the difference is less marked in certain of the gullies. In filled panels, the closure rates in the gullies were twice those measured inside the panels. However, in unfilled panels the closure rates were higher inside the panels than in the gullies.

2.3.4 Response of backfill to seismic damage

The main placement parameters relating to backfill that influence its effectiveness as local support are the stope width, distance to the face of the fill front, percentage area filled and stiffness or porosity (i.e. quality) of the backfill (Squelch, 1993). Underground observations have

indicated that it is essential to maintain the distance of the backfill to the face at less than 6m (Squelch and Gurtunca, 1991), and that backfill of high porosity increases the shrinkage and containment problems. The initial porosity of a backfill determines its stiffness, and therefore its strength, at a given strain. Thus, backfill of low porosity (40 per cent) is stiffer than backfill of high porosity (49 per cent) at the same strain, as shown in Figure 2.3.3. However, from the study of stress-strain behaviour for different backfill materials of different porosities by Squelch (1993), it was found that over the range of strains that usually occurred during the first 15 to 20 m of face advance (the period when local support is required) in stopes with fairly high strain rates, there was little difference in backfill performance for fills with porosities of less than about 41 per cent.

Backfill porosity has, therefore, been found not to be so critical a factor in terms of local support in narrow stopes with high closure rates (i.e. high backfill-strain rates), except for high porosities and unless significant shrinkage occurs. This is because shrinkage delays the onset of load generation and therefore affects the backfill's support of the hangingwall. However, Squelch (1993) has shown that higher porosity lessens the effectiveness of backfill as regional support when the backfill is subjected to high stresses. This is an important aspect if backfill is intended to provide regional support as well as local support. In the case of wide stopes (e.g. wider than 2 m), the backfill needs to be free-standing and to exhibit minimal shrinkage. This is obtained by addition of cementitious binder to the backfill.

To assess the potential of backfill to bring about positive changes in the stability of stope hangingwalls, Squelch (1990) carried out *in situ* stress measurements at various distances into the hangingwall of filled and unfilled stopes. The results of the work are shown in Figure 2.3.4 and Figure 2.3.5. The graphs show that the placement of backfill changes the principal horizontal stresses in the first 2 m of hangingwall strata from being compressive in the perpendicular direction to the stope face and tensile parallel to the face, to being compressive in both directions. That is, in backfilled stopes, the likelihood for falls of ground to occur because of unstable blocks is greatly reduced owing to more effective clamping, which provides more stable ground conditions. The greater aerial support provided by backfill and its greater stiffness compared to conventional pack support reduces rockburst damage (Gay *et al.*, 1988). Jager *et al.* (1987) and Gurtunca *et al.* (1989) also showed that the work done by backfill during rockbursts could be more than three times the work done by conventional support systems.

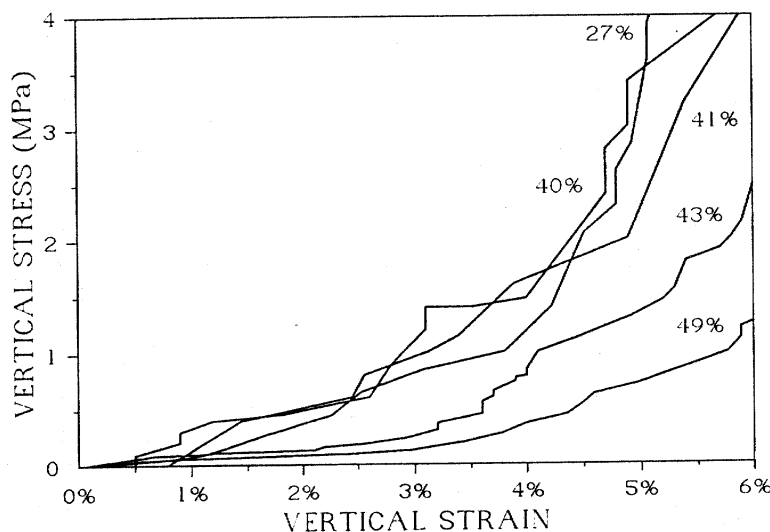


Figure 2.3.3 *In situ* stress-strain curves for backfills of different porosity during the first 15 to 20m face advance after placement in narrow stopes under high closure rates (after Squelch, 1993)

Jager *et al* (1987) studied the relative ability of backfill and other support systems to counteract the kinetic energy imparted to the rock by seismic ground motion. They calculated the average work done by four support systems between the stope face and 15m back, during rapid convergence of 300 mm. It was concluded that the backfill was superior to the other support types in absorbing seismically generated energy.

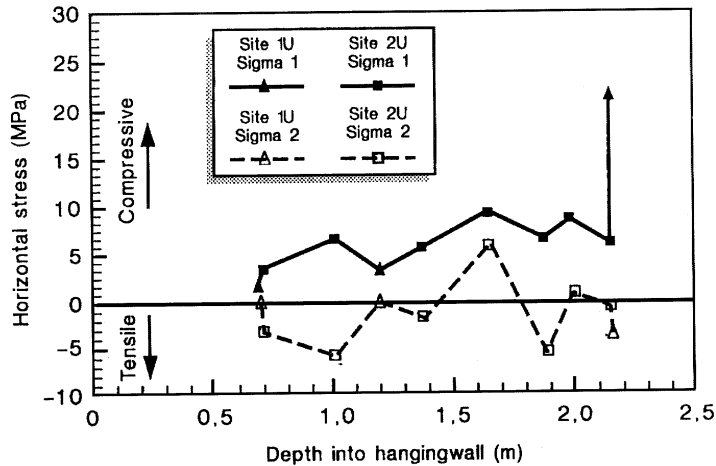


Figure 2.3.4. Horizontal stresses measured in the hangingwalls of unfilled panels (after Squelch, 1990)

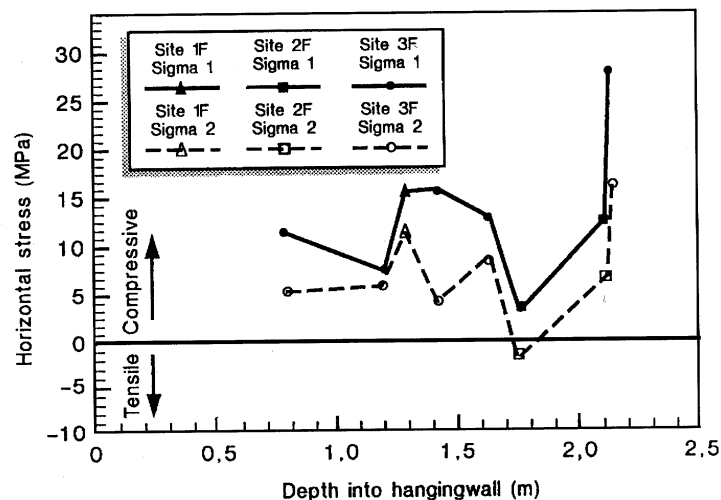


Figure 2.3.5. Horizontal stresses measured in the hangingwalls of backfilled panels (after Squelch, 1990)

In another study of backfill as local and regional support at Vaal Reefs, Macfarlane *et al.* (1998) reported a different experience of backfilling in a scattered mining environment. Contrary to Squelch's (1990) findings, the authors reported that based on back analysis of many collapsed stopes, hangingwall clamping stresses caused damage to the weak and laminated hangingwall rock. The authors concluded that the following differences at Vaal Reefs seem to have limited the potential benefits of backfilling:

- smaller mining spans;
- shallow dip of around 10 degrees;

- high stoping width; and
- relatively weak and laminated hangingwall rock.

The results of the stress-strain response of various backfills to rockburst-induced closure are listed in Table 2.3.1. It was clearly shown that the average work done by the different backfills during a rockburst is considerably higher than can be achieved by any other conventional support system such as timber packs, timber props, hydraulic props, etc. It was calculated by Jager *et al* (1987), that the average work done by a timber pack system close to the stope face is about 43 kJ/m² for 300 mm rapid convergence and 70-100 kJ/m² for hydraulic prop systems. The figures listed in Table 2.3.1 indicate that the maximum work done by the backfill during the rockbursts was 260 kJ/m² and averaged 117 kJ/m² for the six rockbursts for which results were obtained.

Table 2.3.1 The stress-strain response of backfill to rockbursts in the backfill sites

Backfill Type	Distance to face at the time of the event (m)	Closure Increase measured (mm)	Vert. Stress before the event (MPa)	Stress Increase measured (MPa)	Work done (kJ/m ²)	Magnitude of the event
Classified Tailings 45% Porosity	11	Stn 1 - 48	1.3	5.2	180	2.8
		Stn 2 - 46	2.5	6.3	260	
		Stn 3 - 46	0.16	0.11	10	
Classified Tailings 52% Porosity	12	Stn 1 - 37	0.2	3.2	67	2.8
		Stn 2 - 62	0.0	2.8	87	
		Stn 3 - 35	0.045	0.3	7	
Classified Tailings 46% Porosity	25	Stn 1 - 6	15	1	93	1.7
		Stn 3 - 7.5	16	0.5	122	
Classified Tailings 46% Porosity	Panel 1 - 27 Panel 2 - 9.5	30	4.5	3.5	188	2.1
		25	1.6	0.9	103	
Dewatered Tailings 40% Porosity	18	20	0.6	2.2	44	2.1
Comminuted Waste 27% Porosity	52	10	23	2	240	*1.45
						1.85

* Two events occurred on the same day

Further evidence of the increased stability of the hangingwall in backfilled stopes was obtained from the measurements of ground movement reported by Spottiswoode and Churcher (1988) and by Adams *et al.* (1990). These measurements showed that backfill effectively reduces the length of the unsupported hangingwall beam from 30 m in a conventional stope to 10m in a backfilled stope. The result is a beam of greater stiffness, which under seismic conditions, resonates at higher frequencies, thus dissipating the seismic energy more efficiently and reducing the potential for seismically induced damage in the stope-face area. Hemp and Goldbach (1990 and 1993) also found that backfill caused a reduction in peak ground velocities and accelerations, and an associated reduction in the off-reef to on-reef amplification. The results confirmed the earlier work by Spottiswoode and Churcher (1988), in that there is less seismic energy available to do damage at higher frequencies, which are more rapidly attenuated than lower frequencies. Furthermore, Goldbach and Hemp (1990) found that high backfill stresses caused closure of fractures in the rock mass surrounding the stope. This, together with the reduced beam length, caused efficient transmission of energy. Goldbach (1991) has shown that this results in hangingwall vibration times being greatly reduced, and the stope being subjected to less damaging frequencies for significantly shorter periods than a conventional stope would experience.

2.3.5 Design of backfill as local and regional support

Backfill is used on deep level South African gold mines as local (in-stope) and regional (mine-wide) support to reduce rockburst and rockfall problems. The few guidelines that exist on the use of backfill for local support are generally empirically based. Furthermore, there are only general guidelines as to what type of backfill is required to fulfil certain support functions or whether the type of backfill used is even important.

Squelch (1993) presented a methodology for the selection of suitable backfills for use, together with other necessary support, in the stope face area. Figure 2.3.6 shows a flow chart for the identification of design or selection criteria for a suitable backfill for local support purposes.

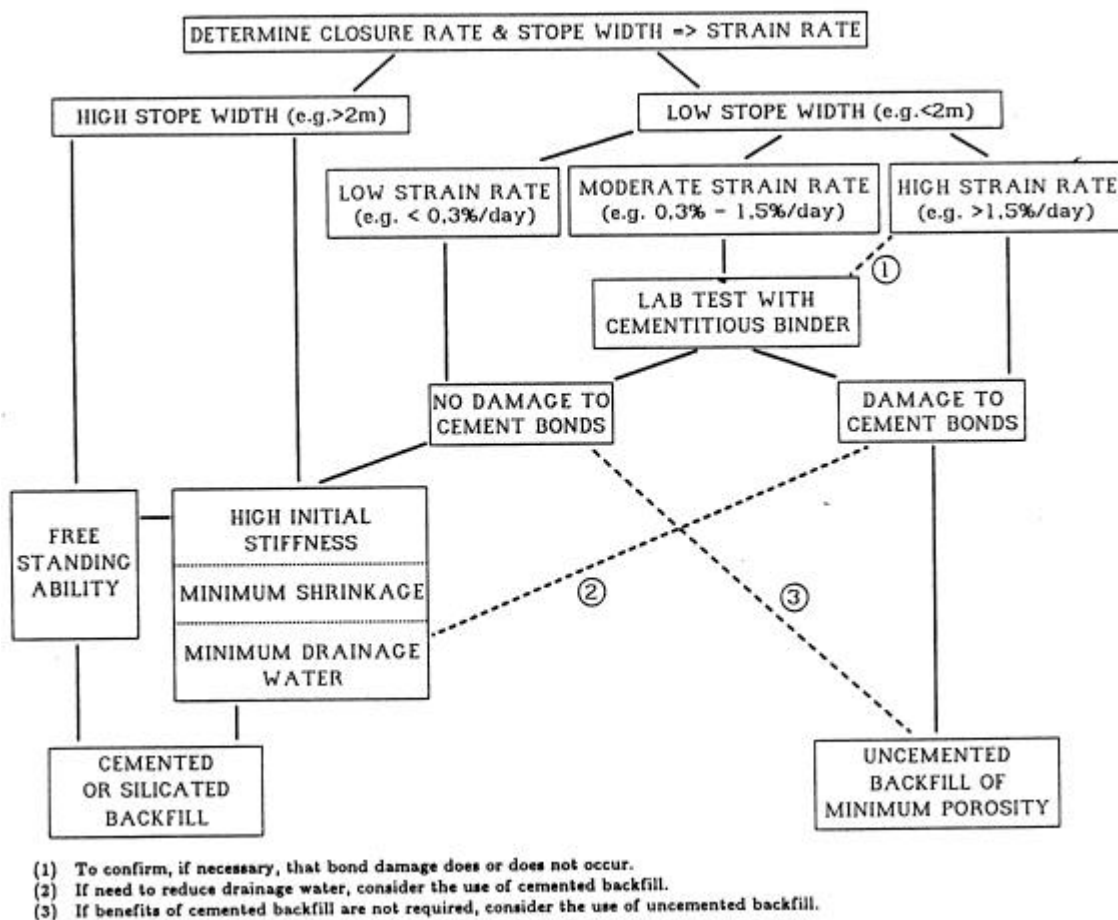


Figure 2.3.6 Design methodology for the use of backfill as local support (Squelch, 1993)

Hodgson and Joughin (1967) established that a relationship existed between rockburst damage, seismicity and the spatial rate of energy release (ERR) in deep mines. Subsequent work has shown that ERR provides a very useful empirical criterion for estimating the quality of strata conditions and the incidence of rockbursts in areas where the geology is relatively uncomplicated. The necessity for regional support and the potential of backfill to reduce the incidence of rockbursts may thus be quantified on the basis of ERR values.

Piper and Ryder (1988) have proposed a flow chart for the design of regional support systems (Figure 2.3.7). It must be emphasised that the proposed design procedure should be used as a guideline only.

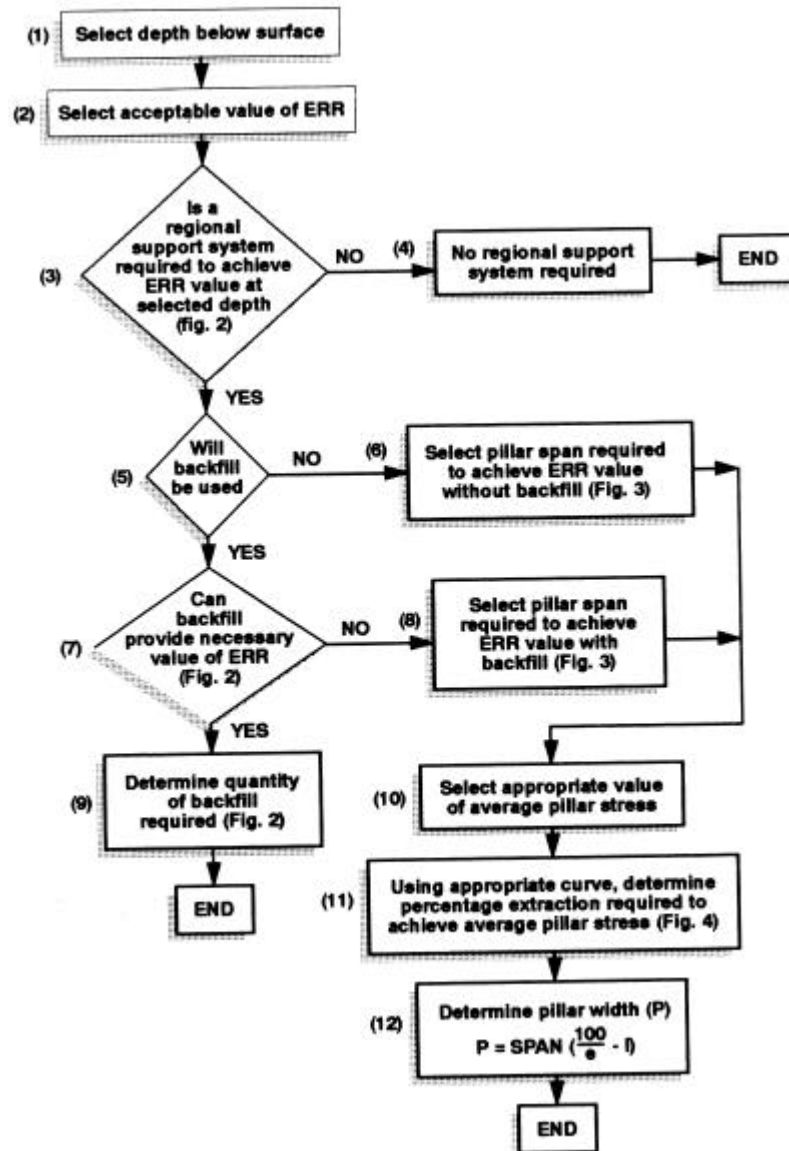


Figure 2.3.7. Flow chart for the design of regional support systems (Piper and Ryder, 1988)

2.3.6 Influence of backfill with and without stabilising pillars

Piper and Ryder (1988) indicated that a regional support strategy is required at depths of greater than 2 km in order to maintain acceptable Energy Release Rate (ERR) levels for large span mining and, further, that significant reduction in the seismic hazard is indicated if backfill is used as regional support. This regional support system may include backfill, rock stabilising pillars or combinations of these elements.

Originally, the only way to reduce longwall vulnerability to rockbursts was to leave strips of unmined reef behind as strike stabilising pillars, which would serve to reduce the stress at the stope faces. Subsequent research and experience have shown that stabilising pillars need not be orientated along strike alone; dip stabilising pillars and bracket pillars can be left to stabilise seismically active geological features. Strike stabilising pillars are the most common pillars used in deep-level longwall gold mines in South Africa at present. Bracket pillars are more common on mines where reef blocks are separated by faults with large displacements, or major igneous

intrusives (e.g. sills and dykes). Dip stabilising pillars are not common as a primary means of regional support, but are being used with great success at Elandsrand Gold Mine, where the Sequential Grid mining system has been employed on the deeper levels since 1991. This section of the review will focus on the influence of backfill with strike stabilising pillars as the primary means of regional support in deep-level mining.

Stabilising pillars have succeeded in their primary aim of reducing the incidence of seismic events and rockbursts. The number of production panels affected by individual, large seismic events has dropped significantly, thereby improving face advance and productivity. One exception has been the small reduction in seismicity and rockbursts casualty rates when 20 m wide pillars were introduced at depths exceeding 3 km. A number of problems have been reported since the introduction of stabilising pillars. The extraction ratio where the stabilising pillars are used is about 80-85 per cent, the remaining 15-20 per cent of the reef being locked up in the pillars. This extraction ratio will have to decrease as mining depths increase, since more reef must be left unmined in order to maintain acceptable levels of ERR. From experience from Western Deep Levels (WDL), where relatively narrow pillars were introduced in 1980 and replaced by wider pillars in 1985 while maintaining 85 per cent extraction ratios, improvements were noticeable but foundation failure persisted in some areas. Seismicity in the vicinity of relatively large geological features still remained a problem. Other problems associated with stabilising pillars were difficulties in certain aspects of mining, such as problems of cutting ventilation and replacement haulage (crosscut) protection slots in established pillars.

These observations have led to a regional support strategy of stabilising pillars in combination with backfill. A major motivation for the use of backfill in rockburst-prone, deep mines is that backfilling gives superior local support while also providing regional support benefits that can complement the current regional-support systems of stabilising and bracket pillars, and also permit increased extraction through a reduction in the size of these pillars. In early work, Gay *et al.* (1988), in evaluating regional-support benefits, concentrated on quantifying the ability of backfill to reduce stope closure sufficiently to give a reduction in energy release rate (ERR), and on monitoring the seismic activity in filled and unfilled panels. The results of this work were inconclusive. Significant reductions in closure were monitored, but the rate of closure never decreased to levels lower than those of elastic closure, nor were there significant differences in seismic-energy release rates between filled and unfilled areas.

Piper and Ryder (1988) quoted a relationship between damage, seismicity and the spatial rate of energy release (ERR) in deep mines. More research by COMRO has shown that ERR provides a very useful empirical criterion for estimating seismicity and, to a lesser extent, the quality of strata conditions and the incidence of rockbursts in areas where geology is relatively uncomplicated. The necessity for regional support and the potential of backfill to reduce the incidence of rockbursts may thus be quantified on the basis of ERR values. Experience with longwall mining at depth suggests that 30 MJ/m² is a typical acceptable value of ERR. The criterion known as Excess Shear Stress (ESS) has also been proposed to assist the design of mine layouts in faulted ground.

The work carried out by COMRO, in evaluating regional support performance of backfill, concentrated on quantifying the ability of backfill to reduce stope closure sufficiently to give a reduction in energy release rate (ERR), and on monitoring the seismic activity in filled and unfilled stopes. Hemp (1993), after an extensive study of the effects of backfill on seismicity patterns, concluded that there was insufficient data to determine whether backfill had an effect on regional seismicity levels. This was due to the difficulties in obtaining *in situ* backfilled and non-backfilled sites that were comparable in terms of geology, geometry, depth and geological features. There was also a shortage of long-term data, which is needed to determine regional

seismic effects of backfill. However, the positive effects of backfill on the local stability of stopes in rockburst conditions were well quantified. These effects are also noted by Jantzen *et al* (1990).

Piper and Ryder (1988) have documented the effect of backfill as regional support by numerical modelling using the MINSIM-D program. An area of 1280 m x 1280 m was modelled using the MINSIM-D program at depths of 2, 3, 4 and 5km to determine the ERR values without backfill. Backfill covering 20, 40 and 80 per cent of the mined area was then introduced into the layout to determine the influence of the quantity of backfill on the values of ERR at each depth. The maximum quantity of backfill that can be placed in a mine was considered to be 80 per cent. The results showed that if 80 per cent of the total mined area is filled with good quality backfill, the ERR could be reduced as much as with stabilising pillars. Figure 2.3.8 shows the results of numerical modelling of influence of quantity of backfill on ERR values with standard modelling parameters. The 'a' and 'b' parameters define the quality of backfill with typical values 5 MPa and 0.3 for a good-quality backfill material. The results also showed a general reduction of the ERR value using good quality backfill and a further slight reduction of ERR from increasing the quantity of backfill to 80 per cent. An ERR value of 40 MJ/m² can be achieved at a depth of 3.3 km only by backfilling 80 per cent of the mined area with good quality backfill. The quality of the backfill material, expressed in terms of its *b* value has an influence on maximum depth of mining with backfill. Sensitivity studies on the influence of quality of backfill on maximum mining depth showed an increase in depth with quality of backfill, maintaining the same ERR at each depth.

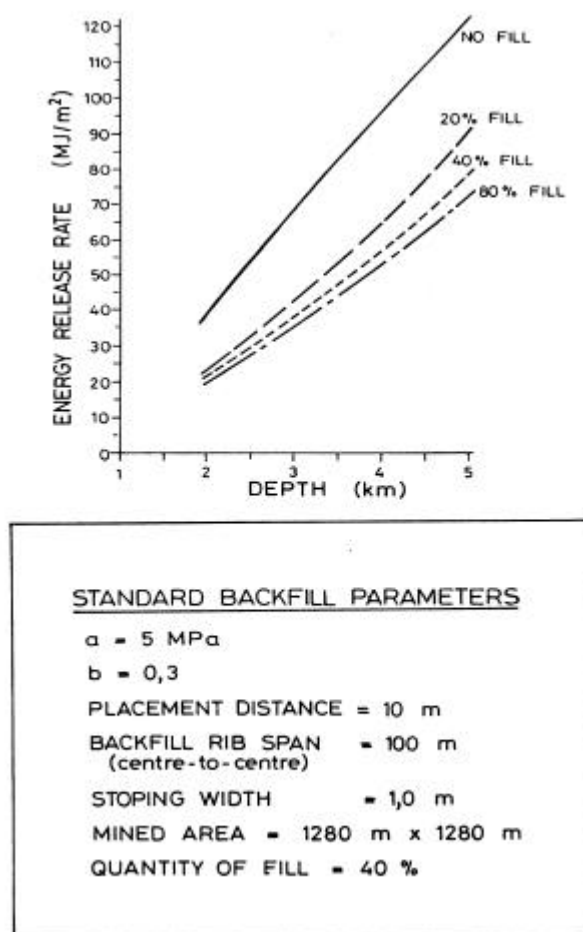


Figure 2.3.8 Influence of quantity of backfill on ERR values for depth of 2 to 5km (Piper and Ryder, 1988)

Results showed that for an ERR value of 30 MJ/m², the maximum depth of mining will be 2.6 km if backfill is placed 10m from the face. However, if placement distance can be reduced to 4m, the maximum depth of mining increases to 2.8 km. The effectiveness of backfill in containing ERR values decreases markedly as the stoping width increases. The results concluded that high quality fill placed to cover at least 40 per cent of the mined area can significantly reduce ERR levels and increase the depth at which acceptably safe mining can be carried out without having to invoke partial extraction methods. Parameters that particularly affect the impact of fill are fill quality and placed fill width, while high stoping widths lead, in general, to higher ERR levels.

2.3.7 Performance of Backfill

2.3.7.1 Performance of Backfill as local support

Backfill reduces the incidences of rockfall accidents in the stope face area, provided that the percentage of backfilling is greater than 60 per cent and the fill-to-face distances are kept to less than 6 m. Moreover, rockburst damage is found to be greatly reduced in backfilled panels, in contrast to that in conventional panels, when the distance between the backfill and the face does not exceed 6 m and good face area support is installed. The following can be stated regarding the performance, design and selection of backfill for local support.

- (i) Backfill reduces damage to the stope face area caused by rockbursts and rockfalls when the fill is kept as close to the face as possible.
- (ii) The closure rates are significantly lower in filled stopes than in unfilled stopes. In filled stopes, high rates of inelastic closure take place only until a vertical stress of 3-4 MPa is generated in the fill. Above 3-4 MPa, the rock mass becomes clamped and displaces into the excavation as a single body.
- (iii) The fill-to-face distance and the interrelated percentage of backfilled area are important factors in obtaining the local support benefit of backfill. The ideal fill-to-face distance has been proved to be 6 m or less and the minimum area backfilled to be 60 per cent of the mined out area.
- (iv) Porosity below a certain value does not have a large influence on the effectiveness of backfill as local support medium, but is always important for regional support.
- (v) To minimise shrinkage, containment, and drainage problems, good-quality backfill of minimum achievable porosity should always be used.
- (vi) The use of cemented backfill is not justified for stope widths less than 1m at depth, and is unlikely to be justified for stope widths of less than 2 m where the strain rate is greater than 1.5 per cent /day, except where drainage water is a problem.
- (vii) Cemented backfill displays low horizontal displacements at low strains resulting in low k-ratios because of the stiffer material. It is required in high stoping widths (i.e. stope widths of 2 m or more) because it provides an immediate and stiffer resistance to closure, high ultimate stiffness, and reduced shrinkage characteristic. It may also be required in multi-reef mining to minimise the deformation of the middling between reefs.
- (viii) Although the initial stiffness of timber packs is similar to that of cemented backfill, the ultimate stiffness of cemented backfill is much higher than that of uncemented backfill and timber packs. This should, in the long term, also make cemented backfill more effective for regional support.
- (ix) Uncemented backfill must be used where strain rates are greater than 1.5 per cent per day for stope widths less than 1 m.

- (x) The area between the backfill and stope face must be supported with appropriate face-area support elements (e.g. rockburst or rockfall props with elongated headboards).
- (xi) The benefits of backfill as a local support result from the reduction of peak particle velocities, vibration times, and the length of the hangingwall beam that can oscillate freely during seismic events (as shown by Hemp and Goldbach, 1993). This explains the observed reduction in rockburst damage and rockfall accidents.
- (xii) A methodology has been proposed by Squelch (1993) for the correct selection of backfill for use as local support in tabular stopes.

2.3.7.2 Performance of backfill as regional support

The results of work done on measurement of backfill as regional support can provide significant benefits such as the following:

- (i) The use of backfill in combination with stabilising pillars will reduce the energy release rate and the pillar stresses, provided that the span between the pillars is sufficient.
- (ii) Underground observations show that, after the introduction of cemented backfill in areas of high stoping width, there is a significant reduction in the incidence of rockburst damage, a decrease in lost blasts due to rockbursts by as much as 80 per cent, and a 30 per cent decrease in rock-related injuries.
- (iii) More work still has to be done to provide a strong argument for using backfill as regional support.
- (iv) The current regional and local support systems can only function as regional or local support whereas backfill, when placed close to the face, serves as local support and later provides regional support. Thus full regional support benefits of backfill will be obtained in deep-level gold mines if backfill is placed in large quantities and in every stope for a long period of time.
- (v) Porosity is always important in regional support effectiveness as opposed to local support effectiveness of backfill.
- (vi) To minimise shrinkage, containment, and drainage problems, good-quality backfill of the minimum achievable porosity should always be used.
- (vii) The benefits of backfill as a regional support have not fully been quantified. The explanation for the failure to quantify a regional-support benefit is that the amount of backfill placed on the gold mines was relatively small in relation to the total area mined. Backfill was mostly used in the mining of difficult areas such as the extraction of shaft pillars, remnants, etc. Recently, some of the gold mines have accepted the mining strategy of placing backfill for local and regional support routinely. However, although mine-wide placement of backfill is being carried out at these mines, the percentage of backfilled areas compared with total area mined is still small.

2.3.7.3 Performance of backfill during seismic loading

Any regional support is generally expected to reduce the magnitude and number of seismic events, together with the frequency of rockbursts and normal falls of ground. Although backfill satisfies most of the above expectations, the effect of backfilling on regional seismicity is not clear at the moment. However, the current regional and local support systems can function only as regional or as local support, whereas backfill, when placed close to the face as possible, serves as local support and later provides regional support.

The benefits of backfill as local support result from the reduction of peak ground velocities, vibration times, and the length of the hangingwall beam that can oscillate freely during seismic events (as shown by Hemp and Goldbach, 1990). This also explains the observed reduction in rockburst damage and rockfall accidents.

The literature review of the use of backfill in South African deep-level gold mining indicates that it has been successful as a local support medium. In preparation for the expected ultra mining depths, the inter-relationship between fill and the face area support with respect to the surrounding rock mass needs to be studied. The overall response of the rock mass to mining is affected by the following three major factors, with their complexity likely to increase with increasing mining depth:

- (i) joints in the rock mass;
- (ii) different geological layers; and
- (iii) inelastic behaviour such as fracture, dilation, bed separation, etc.

2.4 Dynamic site response

The dynamic behaviour of the regional and local support and their interaction under dynamic loading is of prime importance in the scope of this project (GAP 615). This behaviour is also known as a dynamic site response. Measurements made during the past three years by staff at the CSIR Division of Mining Technology GAP 201 (Durrheim *et al.*, 1997) and GAP 530 (Hagan *et al.*, 1998) have shown that the phenomenon of the dynamic site response of the skin of deep-level excavations is real and significant.

Many interesting results have been obtained. For example, it was found that the peak particle velocity on the skin of the excavation may be amplified by four- to ten-fold compared to a point in solid rock a similar distance from the focus (Durrheim *et al.*, 1997). The total energy flux increases by even larger amounts. Ground motion at points less than one metre apart shows differences in amplitude and phase, which can only be accounted for by large strain across fractures (Spottiswoode *et al.*, 1997). A case has been recorded where this behaviour changed dramatically following a nearby seismic event, suggesting that a significant change in stress occurred. Two mechanisms of dynamic response have been recognised from 2D and 3D seismic measurement: structural response, defined as a common spectral behaviour at all surface seismograms; and local site effect, defined by spectral peaks at one or two surface seismograms (Milev *et al.*, 1999).

The study of dynamic site response was done in combination with a comprehensive investigation of rockbursts that have caused damage and posed a hazard to workers. Twenty eight accident site investigations were completed in the years 1994 to 1997, inclusive, mainly as part of project GAP 201. Six additional investigations were done in 1998 as part of GAP 530 and, in combination with the initial findings, these have served to highlight certain aspects of the rockburst phenomenon. These aspects include the existence of the following problem areas that need to be addressed:

- a lack of knowledge of the stress fields affecting a particular mine;
- an underestimation of the extent of the unstable zone surrounding stopes;
- poor condition of support elements;
- siting of stopes in areas of fault loss, and stopes intersecting faults at oblique angles;

- ineffective gully support design and implementation;
- shape of remnant pillars; and
- backfill usage.

The source mechanism in the vast majority of cases was diagnosed as a seismic event resulting from slip on either a dyke contact or a fault. In some cases the seismic data coupled with the extent, nature and location of the damage make the degree of certainty, with respect to the source mechanism involved, extremely high.

A significant input in the understanding of dynamic site response, rockburst damage and support behaviour was done in GAP 530 by simulating a seismic event in solid rock close to an excavation. The event was simulated by means of a large blast detonated in solid rock close to a crosscut sidewall. The experiment involved a design of seismic source and fairly dense seismic observations in near and far field, high-speed video filming, and study of rock mass condition (fractures, joints, rock strength etc.). Knowledge of the site conditions before and after the experiment was also gained. Some of the important findings are listed below:

- The distance of the blast holes from the tunnel wall ensured that no gas pressure was directly involved in damaging the wall of the tunnel.
- Two areas of damage were identified on the blasted sidewall wall: (i) area with high intensity damage: ground velocity at 3.3 m/s was recorded by an accelerometer ejected with a block of rock; (ii) area with low intensity damage: ground velocity at 1.6 m/s was recorded by an accelerometer which remained in the tunnel wall.
- High-speed filming revealed rock fragments being ejected from the wall at velocities in the range of 0.7 m/s to 2.5 m/s. The measurements were taken in the area of low-intensity damage.
- The attenuation of maximum velocities for the main blast as a function of distance R was found to follow the law of $1/R^{1.7}$, in the near field (6 m to 30 m).
- The simulated rockburst was recorded by the Vaal River Operations regional seismic network with magnitude estimated as $M_L = 1.3$.
- Peak particle velocities measured on the blasted sidewall wall after the blast were amplified some five to six times compared to the peak particle velocities measured before the blast.

2.5 Current support practice adjacent to regional support

A review of current codes of practice and a series of interviews with mine rock mechanics personnel determined that, as a general rule, no special design strategies existed for the installation of in-stope support adjacent to regional support. In the event that poor ground conditions were encountered or developed after time the mining personnel's response is to install additional support units of a type and a density dictated by circumstances and guided by experience.

2.6 Discussion and Conclusion

Considerable work has been carried out on various aspects concerning regional support systems (stabilising pillars with or without backfill), local support systems in general, and for a specific reef. However, the general influence of regional support on local stope conditions and support is not well understood and a number of anomalous mining conditions have been reported in the vicinity of regional pillars, some positive and some negative. Mining in similar rock conditions has apparently resulted in very different conditions, depending on the type of regional support used in the region. This has shown that little work has been conducted to determine specifically the influence of regional support systems on local support requirements or design.

Significant investigations have been conducted on pillar behaviour, pillar failure, fracturing in and around pillars and geological influences on pillar behaviour. All these findings will be considered in the current investigation.

Research requirements for the current project are identified as the following:

- an improved understanding of the inelastic behaviour of the rock mass in order to further understand stabilising pillar failure and its effect on the rock mass conditions in their vicinity;
- a detailed analysis and quantification of fracturing and deformation mechanisms in and around pillars; and
- an understanding of the influence of local geology and structure on pillar behaviour.

In order to design local support, it is necessary to determine the zone of influence and the behaviour and characteristics of the regional support. Therefore, the above findings should be incorporated into a support design methodology for the local support in this region.

Backfill has been studied extensively as a local and regional support by a number of researchers. This includes the correct placement of backfill to maximise the regional benefits, placement properties of effective backfills, their *in situ* behaviour and the benefits of backfill placement on a local and regional scale.

Previous research into the influence of backfill as a support medium has only quantified the local support effects of backfill. Although a number of attempts to quantify the regional support effects of backfilling have been made, the results were inconclusive. Further research into backfill in terms of measurements and numerical modelling needs to be done to study the influence of backfill as a regional support on the stope working area and on gully conditions. The current regional and local support systems can function only as regional or as local support, whereas backfill, when placed close to the face, serves as local support and later provides regional support. This is an important advantage to be gained from backfilling on the gold mines. Both these benefits are gained to varying degrees in all mining environments where any backfill is used and properly placed, with one possible exception – where the immediate hangingwall is well bedded with bedding partings spaced less than about 0.4 m apart. This possible exception will be a focus area of this project.

The information and experience obtained from *in situ* monitoring of backfill performance and rock mass response together with numerical modelling work can be used to quantify the benefits of backfilling as a regional support medium. This work will also assist in determining the local support requirements in the vicinity of backfill.

3 Research methodology

The research methodology comprises a combined approach of underground site investigations and numerical modelling, according to which various effects of the regional support elements, i.e. backfill and pillars, are studied. Aspects studied include:

- variation in ground motion (i.e. PPV) and stope closure in relation to distance from regional support and stope face;
- variation in hangingwall condition, e.g. hangingwall profile and fracture intensity;
- installed support quality; and
- numerical modelling of regional effects.

3.1 Review of installed support

The type and layout of installed support components at each site is recorded as is any obvious comment regarding their quality and effectiveness.

3.2 Hangingwall condition

Hangingwall condition, in general, is one of the most useful indications of the success or failure of a particular mining or support strategy. This means that hangingwall conditions can change considerably by the introduction of a particular mining or support strategy. It is thus necessary to quantify these differences. In this work, an attempt has been made to quantify hangingwall roughness in the vicinity of regional support in stopes and then compare the results with those obtained after a period of time. Hangingwall profiling was used to quantify the differences, which can be considered as a measure of stability as well as a way of determining the area of influence of the regional support. In addition, a hangingwall rock mass classification exercise was conducted at discrete locations along gullies at several sites to assess time-dependent deterioration of the hangingwall.

3.2.1 Hangingwall profiles

Profiles were determined by stretching out a measuring tape over a particular length in the strike gully and measuring the distance between the tape and the hangingwall at various points along the tape, i.e. the peaks and troughs of the hangingwall above the tape are measured (see Figure 3.2.1). The dip of the tape and average dip of the hangingwall are recorded. Three methods are employed to analyse the results and quantify the hangingwall roughness. These are the cumulative percentage of size of profile steps, the profile length and the average gradient methods (after Grodner, 2000).

3.2.1.1 Cumulative percentage of size of profile steps

Profiles are normalised for differences in dip between the tape and the hangingwall. Each normalised profile is then zeroed by subtracting the minimum y value (height), thereby allowing various profiles to be compared. A cumulative percentage plot is then drawn up of the heights of the steps in the profile. According to Grodner (2000), this method is useful only when the dominant sizes of the steps in the various areas are significantly different and, hence, show up as a distinct crest on the cumulative curve.

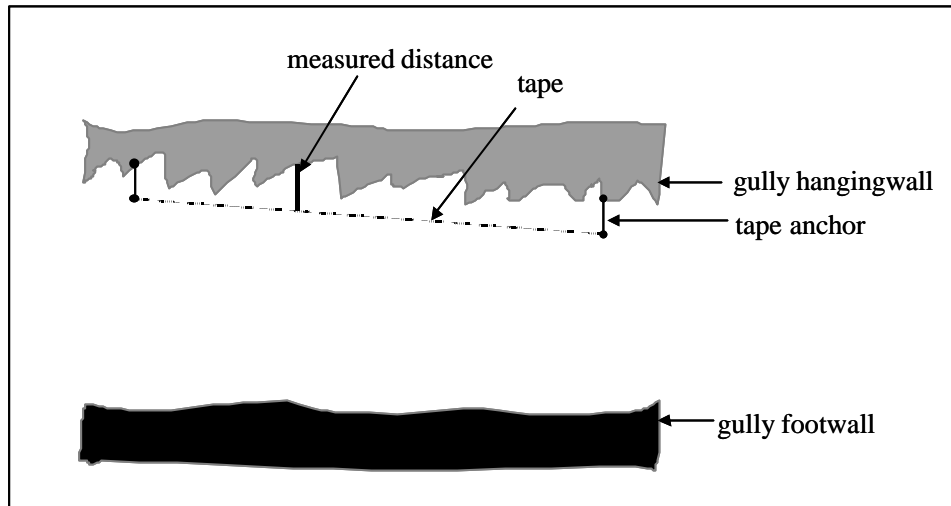


Figure 3.2.1 Schematic of set-up and method of hangingwall profiling

3.2.1.2 Profile length

In this technique, the total length of each profile is determined by calculating the distance between successive points along the profile and summing these distances (Equation 1).

$$Length = \sum_{i=1}^{n-1} \sqrt{(x_i - x_{i+1})^2 + (y_i - y_{i+1})^2} \quad (1)$$

where:

n = number of points,

x_i, y_i = coordinates of the i th point, and

x_{i+1}, y_{i+1} = coordinates of point $i+1$

The straight-line distance between the first and final points is subtracted from the summed length. The smaller this final value, the smoother the hangingwall, therefore a quantitative description of the condition of the hangingwall can be obtained.

3.2.1.3 Average gradient

With this method, the greater the difference in heights between adjacent points on a profile, the less smooth the profile is. Therefore, a measurement of the change in gradient between points along a profile provides an indication of the roughness of the profile. Gradients (cf. Figure 3.2.2) are calculated by dividing the absolute difference in x-coordinates by the absolute difference in y-coordinates and taking the average of these for the entire profile.

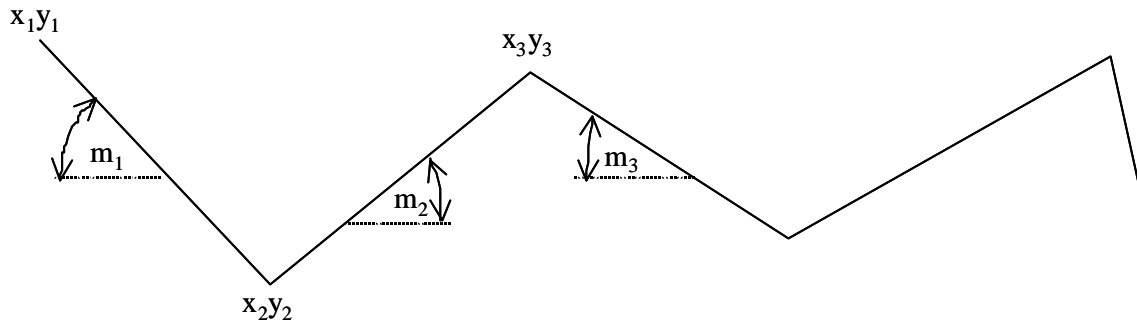


Figure 3.2.2 Sketch of method used in computing the gradients of a profile

3.2.2 Hangingwall quality

Rock mass ratings surveys were carried out on the hangingwall of strike gullies in the vicinity of several of the ground motion and closure instrumentation sites. The aim of the exercise was to quantify gully hangingwall condition at various distances back from the face, and thereby determine if time-dependent and/or positional (in relation to regional support) deterioration of the hangingwall occurs.

3.2.2.1 Rock mass rating assessment

The fractures resulting from blasting and high stress were included in the analysis and treated as the most significant of the discontinuities in the rock mass ratings. Survey windows (generally 5 m in length) were selected approximately 35 m apart in the strike gullies where instrumentation sites were located and in gullies above and below these locations. The first and last windows in each survey were sited near the face or dip pillar and the centre gully respectively, usually with one window midway between these sites. Only discontinuities within the window were logged. In particular, persistency, weathering and frequency of the discontinuities and fractures were compared to the same sets in other windows. Rock quality designation (RQD) was measured by assuming that if a horizontal borehole were drilled in the hangingwall perpendicularly to the fracturing, breakages in the core would occur at every discontinuity. Thus all solid pieces greater than 100 mm were added together and divided by the length of the window. In one case, where the fracturing was parallel to the gully, the RQD was divided by the width of the gully.

Three rock mass rating systems were compared in the analysis, these were 'RQD', 'RMR' (Bieniawski) and 'Q' (Barton *et al.*). To eliminate human judgement, only the geological influences on the discontinuities were assessed in the analysis, thus adjustments for discontinuity orientation and stress were not used. Under these conditions 'Q' is known as Q' (Q-prime) and 'RMR' as RMR' (RMR-prime).

Figure 3.2.3 was developed from a set of tables converting 'JRC' to 'Jr' values for Barton's 'Q' rating system. The graph is used to convert offset measurements, made from a 0.5 m long straight edge placed along a discontinuity surface underground, to a joint roughness value.

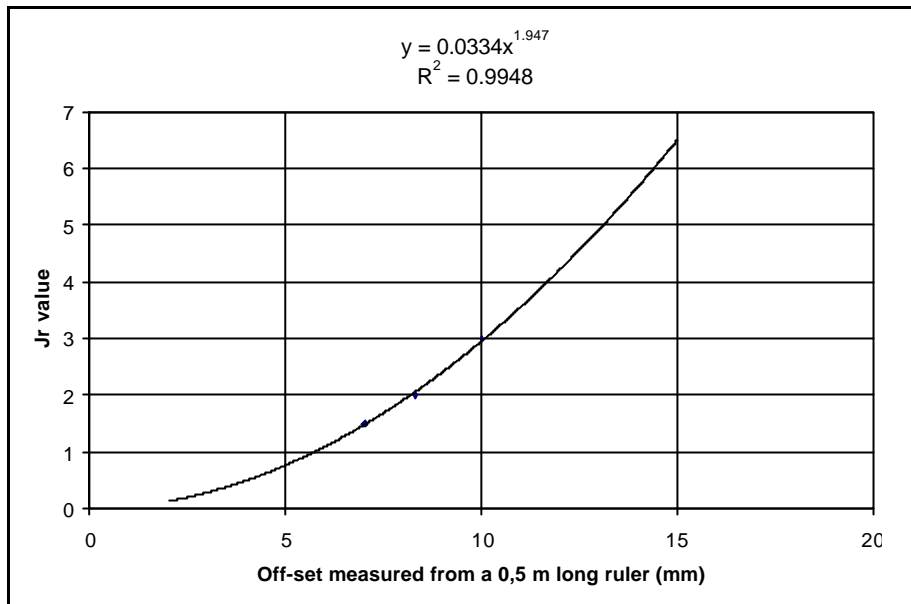


Figure 3.2.3 Graph of offsets measured from a 0.5 m long straight edge compared to Barton's 'Jr'-value for the 'Q' rating system

3.3 Ground motion and stope closure

In order to fulfil the underground monitoring programme, an integrated system was developed for monitoring quasi-static and dynamic movements. The system consists of the following components (Figure 3.3.1):

- Ground Motion Monitor – existing unit with well proven reliability
- Interface Quasi-Static Box – new development
- Quasi-static transducers (strain gauges and/or closure meters)
- Dynamic transducers (uniaxial and triaxial geophones)

As part of the study of the effect of backfill on the peak particle velocities and amount of closure, underground sites at selected mines were instrumented with a system supporting quasi-static and dynamic transducers and consisting of two continuous closure meters and five geophones. (Figure 3.3.2) The first closure meter, CL 1, was installed close to the face and the second closure, meter CL 2, was installed in the back area. Geophone G 1 was installed closest to the face on the hangingwall. Geophones G 2 and G 5 were placed close to closure meter CL 1 and CL 2 respectively, and geophones G 3 and G 4 installed between the closure meters. A typical installed configuration using this integrated system is shown schematically in Figure 3.3.2

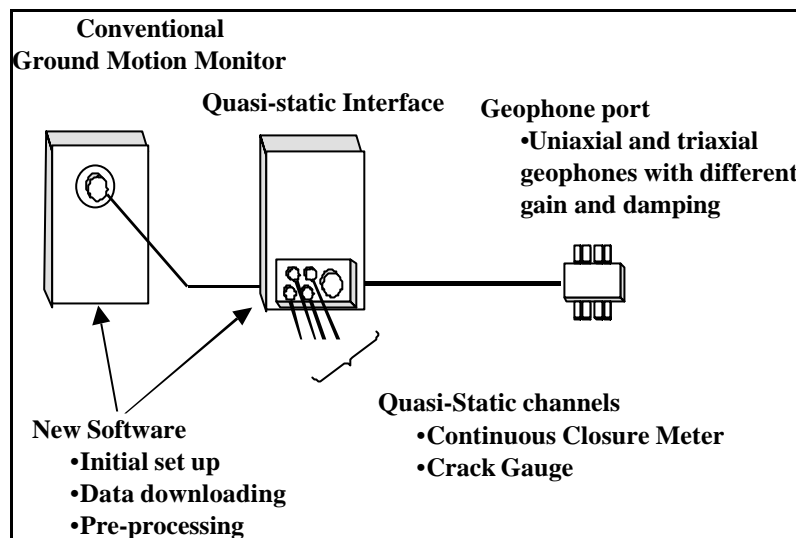


Figure 3.3.1 Schematic representation of the interaction of the quasi-static and dynamic part of the integrated measuring system

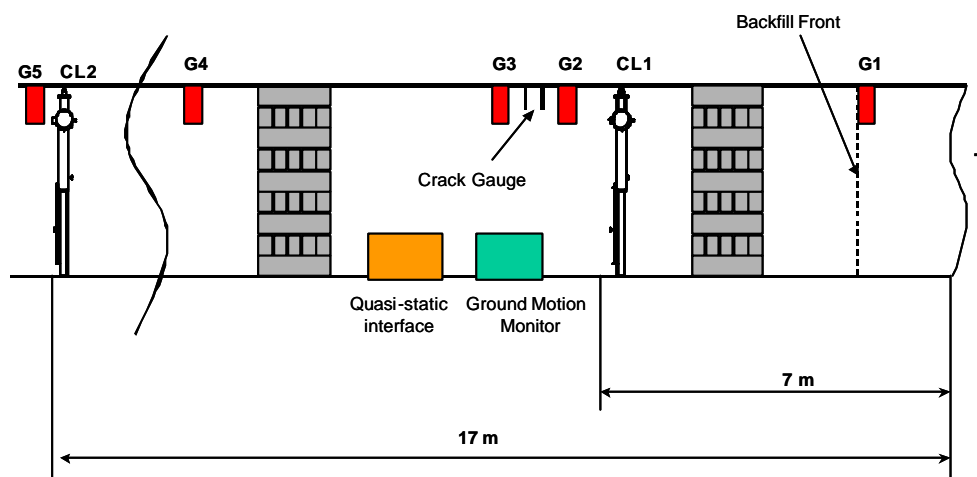


Figure 3.3.2 Schematic of ground motion and closure instrumentation configuration

An additional underground experiment using a crack gauge was included at one monitoring site (e.g. Kopanang mine) to study the stope hangingwall stability in respect of stresses acting in the hangingwall. The crack deformation at a point has three components of motion. The first of these is normal (opening or closing), which varies with changes in the skin stress. A crack opening could indicate increased possibility of hangingwall instability. The second is dip-slip shear, where the fallout of one block is an extreme case. The third component is strike-slip shear.

For this study a crack gauge was installed across a crack running parallel to the stope face. Two geophones were also installed in close proximity to the crack gauge to compare the change in PPVs with respect to whether the crack is opened or closed. Figure 3.3.2 indicates the integration of the crack gauge with the ground motion and closure instrumentation and Figure 3.3.3 shows the actual underground installation of crack gauge and geophones.

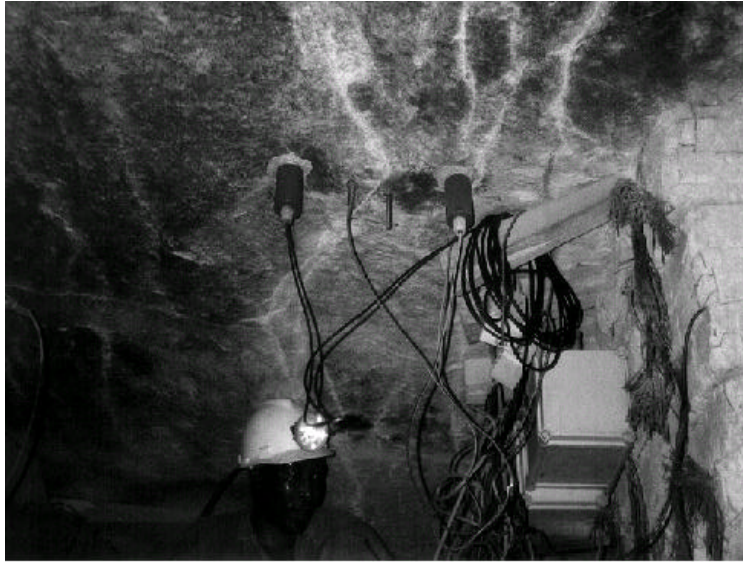


Figure 3.3.3 Photograph showing installed crack gauge and associated geophones

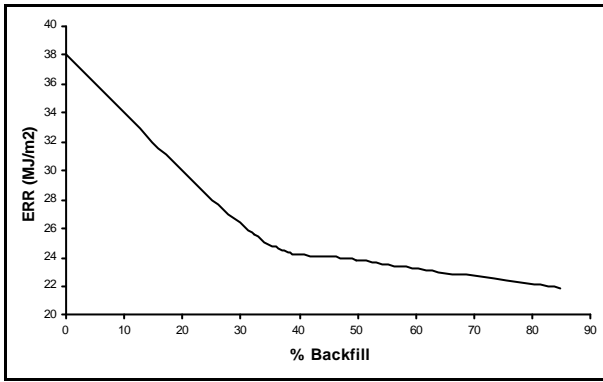
3.4 Numerical modelling

Numerical modelling was undertaken to complement and assist with the interpretation of data where necessary. As well as modelling of the actual underground geometries associated with monitoring sites, a series of theoretically based elastic modelling runs was conducted to provide generic insights and background to regional pillar support and backfill interaction.

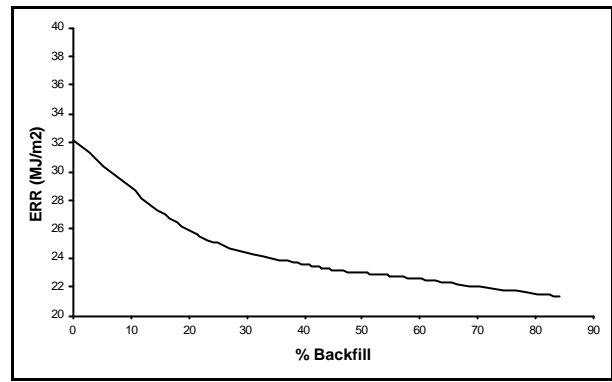
For the generic cases, two different regional pillar systems were modelled, strike stabilising and dip stabilising pillars. In these runs, the constant parameters are depth (3200 m), k-ratio (0.5), Young's modulus (50 GPa), Poisson's ratio (0.2), mining height (1 m) and, where applicable, backfill to face distance (5 m). Whereas the parameters of span, pillar width, and face advance rate are 240 m (on dip), 40 m and 10 m, respectively, for the strike pillar cases; and 203 m (on strike), 30 m, and 15 m, respectively, for the dip pillar cases. The percentage backfill in the mined out region was varied from 0 per cent (i.e. unfilled) to 84 per cent.

The criteria used in assessing the results in terms of regional support effects are: average energy release rate (ERR) at the faces (i.e. average of all panels along a stope), average pillar stress (APS), average face stress and average closure.

Analysis of the results reveals that when 60 per cent of the stope is backfilled, the average ERR (Figure 3.4.1) is reduced by almost 40 per cent (i.e. from 38 MJ/m² to 23.3 MJ/m²) in the strike stabilising scenario and about 30 per cent (i.e. 32 MJ/m² to 22.5 MJ/m²) in the dip stabilising scenario compared to the unfilled (0 per cent fill) case. Likewise, the APS values (Figure 3.4.2) for a 60 per cent fill ratio decreased by around 30 per cent in the strike stabilising scenario and about 20 per cent in the dip stabilising scenario compared to the unfilled case. Similar analyses show a 25 per cent and 18 per cent reduction in average face stress (Figure 3.4.3) and 45 per cent and 29 per cent decrease in average closure for the strike stabilising and the dip stabilising scenarios, respectively.

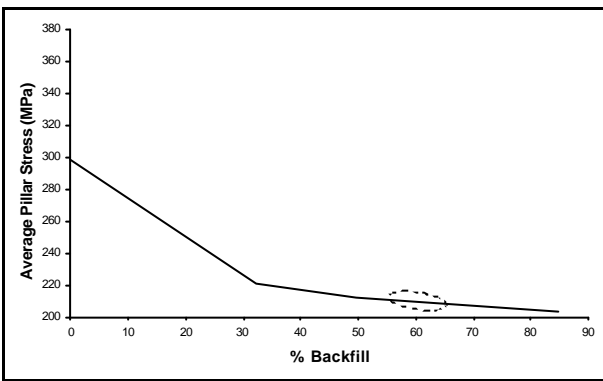


(a)

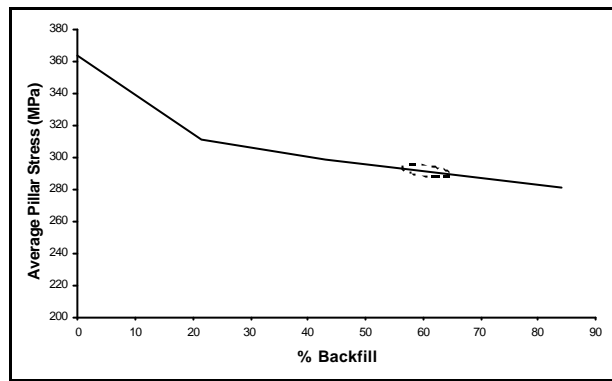


(b)

Figure 3.4.1 Average ERR as a function of percentage backfill for (a) strike stabilising pillar, (b) dip stabilising pillar geometries

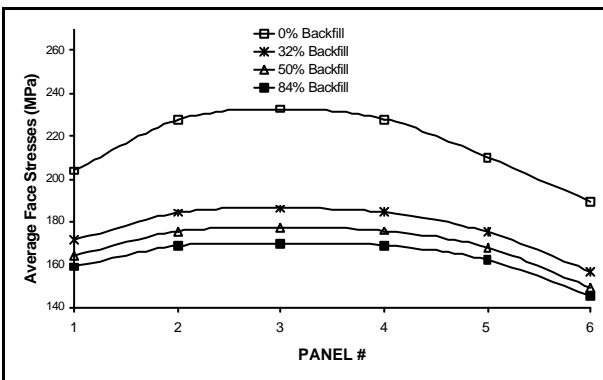


(a)

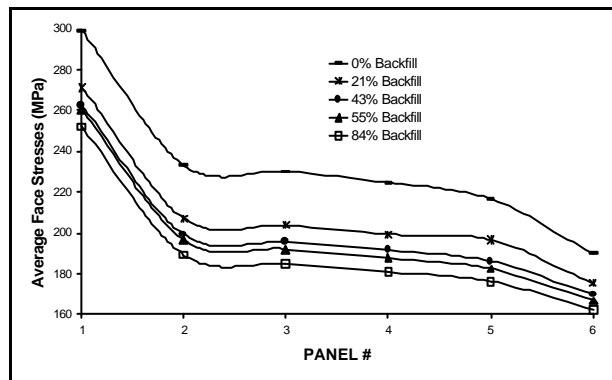


(b)

Figure 3.4.2 Average pillar stress as a function of percentage backfill for (a) strike stabilising pillar, (b) dip stabilising pillar geometries



(a)



(b)

Figure 3.4.3 Average face stress for each panel as a function of percentage backfill for (a) strike stabilising pillar, (b) dip stabilising pillar geometries

In addition, Figure 3.4.4 shows the changes in major principal stress along strike and dip stabilising pillars as a function of percentage backfill.

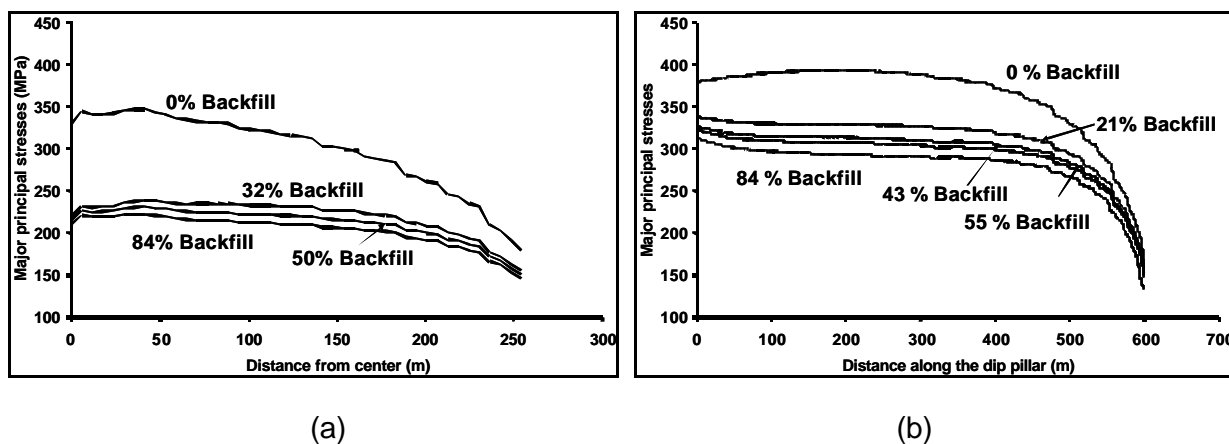


Figure 3.4.4 Major principal stresses along the pillars as a function of percentage backfill for (a) strike stabilising pillar, (b) dip stabilising pillar geometries

Modelling of the selected underground sites was carried out to complement and assist with interpretation of underground results, such as the build-up of stress in regional pillars and its association with ground motion findings.

4 Site investigations

To meet the objectives of this project, various mines were selected for monitoring so as to include a diversity of reef horizons and regional support layouts. The mine sites are:

- Kopanang mine: Vaal Reef with scattered mining layout and backfill
- Tau Tona: Carbon Leader reef with strike stabilising pillar layout with backfill and no backfill
- Deelkraal: VCR reef with strike stabilising pillar layout and backfill
- Mponeng: VCR reef with sequential grid, dip pillar layout without backfill
- Driefontein 5 Shaft: Carbon Leader with dip stabilising pillar layout with backfill
- Driefontein 1 Shaft: VCR with strike stabilising pillar layout with backfill

At each site a variety of monitoring and research exercises were undertaken, and these include:

- ground motion (i.e. PPV) measurements;
- closure measurements;
- hangingwall condition, e.g. profiling and fracture mapping;
- numerical modelling;
- backfill parameters (where relevant); and
- rock parameters.

The findings of the underground investigations are presented on a site-by-site basis. It must be noted, however, that the lack of availability of suitable and representative monitoring sites or the stopping of mining as sites were being instrumented was a major hindrance during the term of the project.

4.1 Kopanang mine

One backfill site and one unfilled site were monitored at a depth of approximately 2500 m at the Kopanang mine, Vaal River Operations, in 64 SW1 raises 17 and 18 (Figure 4.1.1), representing the 'scattered mining with backfill and scattered mining without backfill' environments, respectively. Panels along the 17 raise line are backfilled while panels along the 18 raise line are supported on elongates. Stopping width is 1.2 m and dip 12 degrees. The hangingwall comprises well bedded quartzite separated by weak argillaceous partings.

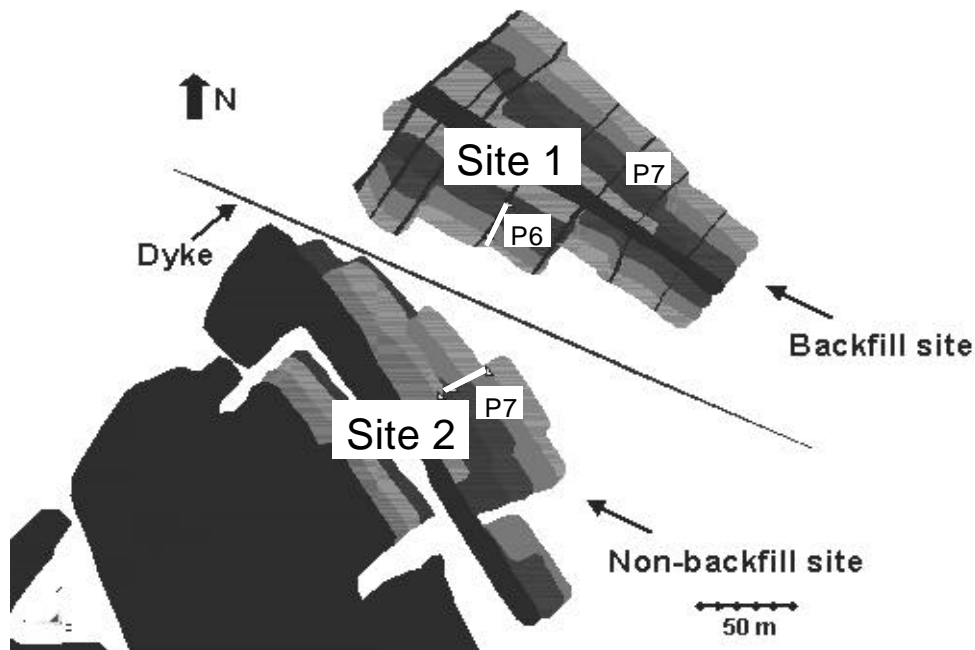


Figure 4.1.1 Schematic of Kopanang mine underground monitoring site layout

The following activities were conducted at the sites:

- fracture mapping;
- hangingwall profile measurements;
- ground motion and closure monitoring; and
- crack deformation monitoring.

4.1.1 Review of installed support

The stope on the southern side of the dyke (64 SW1-18) was supported by conventional elongate support, while the stope on the northern side of the dyke (64 SW1-17) was backfilled. The area of the stope that was filled is between 50 per cent and 60 per cent. Ground condition in the monitored backfill panel (64 SW1-17 P6) was noted as good with a small amount of bed separation and fractures 150 mm apart. Conversely, in the monitored unfilled panel (64 SW1-18 P7) the ground condition was recorded as bad, the hangingwall highly fractured and footwall heave with elongates punching into the footwall.

4.1.1.1 Conventional support

The unfilled panels (64 SW1-18) are supported with pre-stressed elongates (Hercules. props), at spacings of 1.5 m on strike and 1.2 m on dip, and temporary face support of 150 mm diameter mine poles, spaced at 1.5 m on dip. In addition, a breaker line of 0.6 m x 0.3 m Durapaks is installed together with or in place of a seventh line of elongates. Gully support consists of pre-stressed Durapaks (0.9 m x 0.6 m) on both the north and south gully shoulders, and 1.2 m rock bolts on a 2x1x2 diamond pattern spaced 1 m on strike, in the gully hangingwall.

The backfilled panels (64 SW1-17) are supported with pre-stressed mine poles at spacings of 1.5 m on strike and 1.2 m on dip, with backfill placed between the rows. Gullies are supported as per unfilled panels.

4.1.1.2 Backfill

A good-quality backfill of a minimum porosity of 41 per cent, and *in situ* porosity of 46 per cent, is placed between the rows of pre-stressed elongates at observed distances of 5.5 m (P6) and 4 m (P7) from the face before the blast. The coefficient of permeability of this backfill is 6.6×10^{-4} . The material is well classified with an ultra-fines content of 6.5 per cent passing 0.010 mm. Moisture content is in a close range of between 18 per cent and 20 per cent, signifying backfill densities of about 2.0 and 2.1 kg/l and, therefore, has a desirable consolidation characteristic.

At the time of backfill sampling it was further recorded that very few sticks were installed between the backfill and the face and that the distance from the front line of support to the face was 4 m. The condition of the hangingwall was nonetheless described as good, with no observed unravelling, despite being highly discontinuous with steep dipping fractures parallel to the face intersected by sets of strike-orientated joints.

4.1.2 Hangingwall condition

The fracture mapping, hangingwall profile measurements and fall of ground analysis conducted at the Kopanang mine site provided the following insights:

- The hangingwall of the Vaal Reef is a glassy, siliceous quartzite with well-developed argillaceous partings. The argillaceous parting influences hangingwall conditions in the face area.
- The hangingwall surface may range from smooth to rough, depending on where the hangingwall is established. Establishment on an argillaceous parting results in a smooth surface (Figure 4.1.2).
- If the argillaceous parting is broken, the hangingwall is typically rough with mining-induced fractures visible (Figure 4.1.3).
- Backfill placement was irregular as sometimes gaps were left between bags (Figure 4.1.4). Inconsistent placement may adversely influence strata conditions at the face.
- Generally, there appears to be a larger variation in the dips of fractures in backfilled panels (45° to 75°) than in those in conventionally supported panels (60° to 75°).
- Fracture spacing was found to be more variable in backfilled panels.
- Poor drilling control in the conventionally supported panels results in poorer hangingwall conditions. There is, consequently, more variability in hangingwall profiles in conventionally supported panels.



Figure 4.1.2 *Smooth hangingwall conditions in a backfilled stope at Kopanang mine*



Figure 4.1.3 *Rough hangingwall conditions in a backfilled stope at Kopanang mine*



Figure 4.1.4 Variability of backfill placement at Kopanang mine, e.g. 1.5 m-wide gap left between backfill

4.1.3 Ground motion and closure

Figure 4.1.5 illustrates the closure rate in the backfilled panel measured close to the face and far from the face (17 m to 21 m). The average closure rate measured close to the face is 2.3 mm/day, while the average closure rate measured in the back area is 2.0 mm/day. A reduction is also recorded in terms of peak particle velocity, which is higher close to the face than in the back area (Figure 4.1.6).

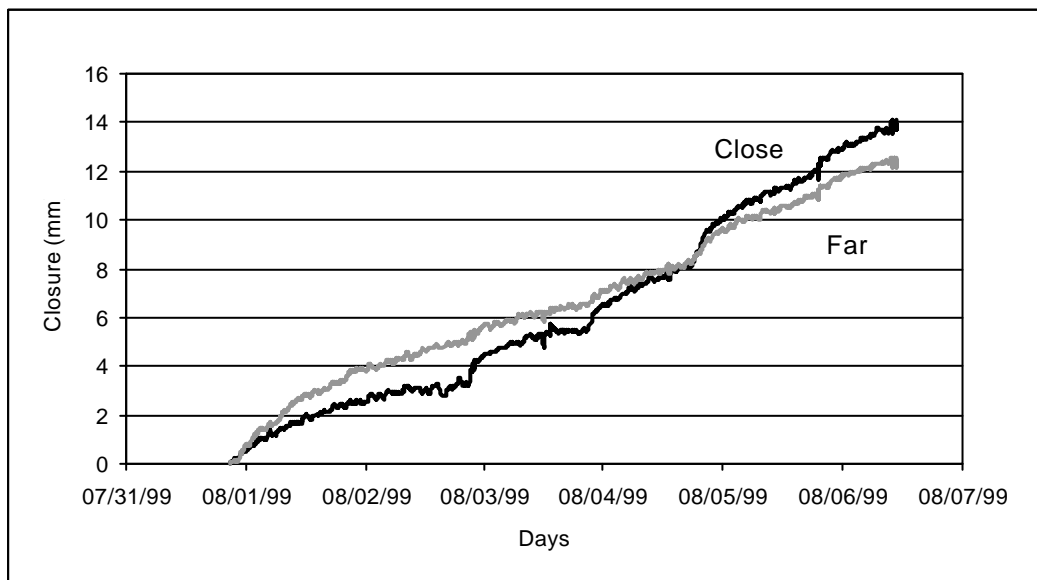


Figure 4.1.5 Closure measurements in backfilled stope at Kopanang mine

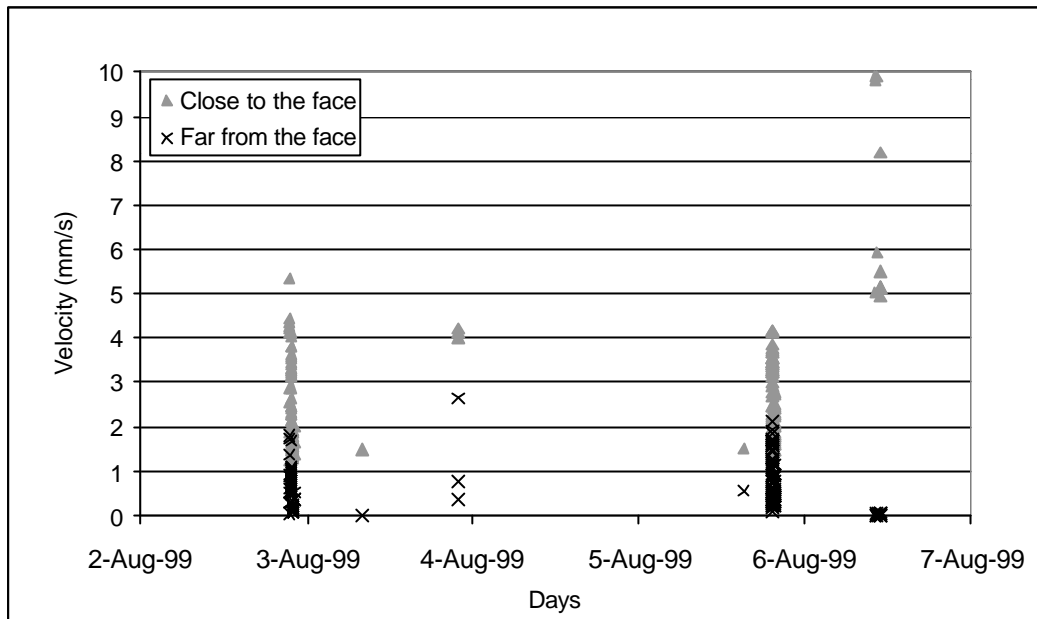


Figure 4.1.6 Recorded peak particle velocities in backfilled stope at Kopanang mine

During the earliest stage, when the backfill is still soft, the rate of closure and peak particle velocities have higher values compared to the rate of closure and peak particle velocities measured far from the face, where the backfill is more compressed. It indicates that interaction between the hangingwall and the backfill improves with distance to the face and time, and that the support provided by the backfill becomes more effective.

A similar monitoring configuration was installed in the non-backfilled site on the same level. The closure rate measured at this site is shown in Figure 4.1.7 and the recorded PPVs are shown on Figure 4.1.8. The closure rate of 9.4 mm/day was obtained close to the face and a closure rate of 6.7 mm/day was obtained far from the face. These values are 3 to 4 times higher than the closure rates measured at the backfilled site.

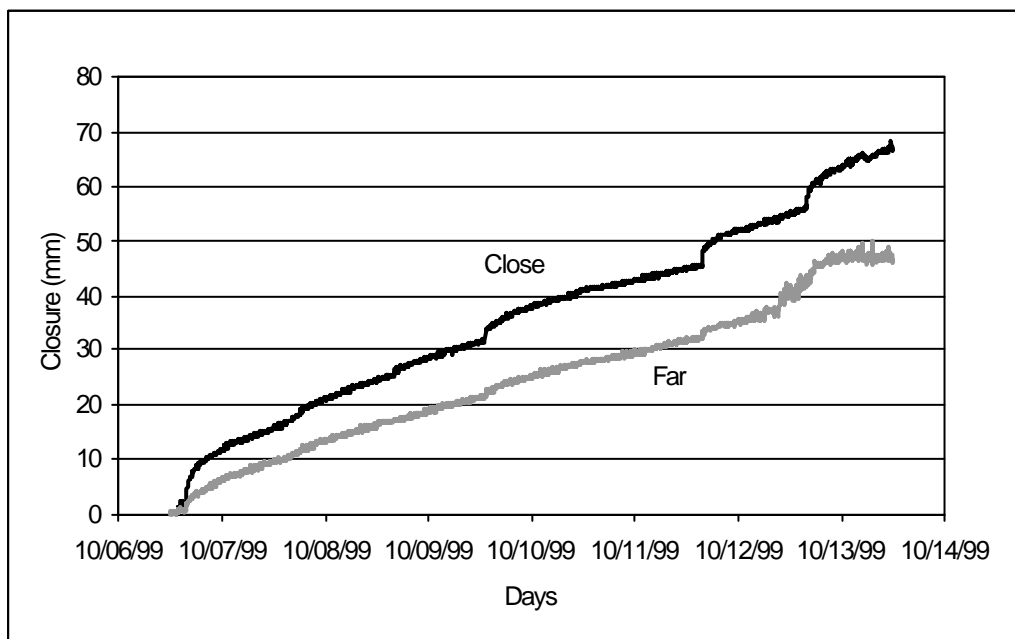


Figure 4.1.7 Closure measurements in non-backfilled stope at Kopanang mine

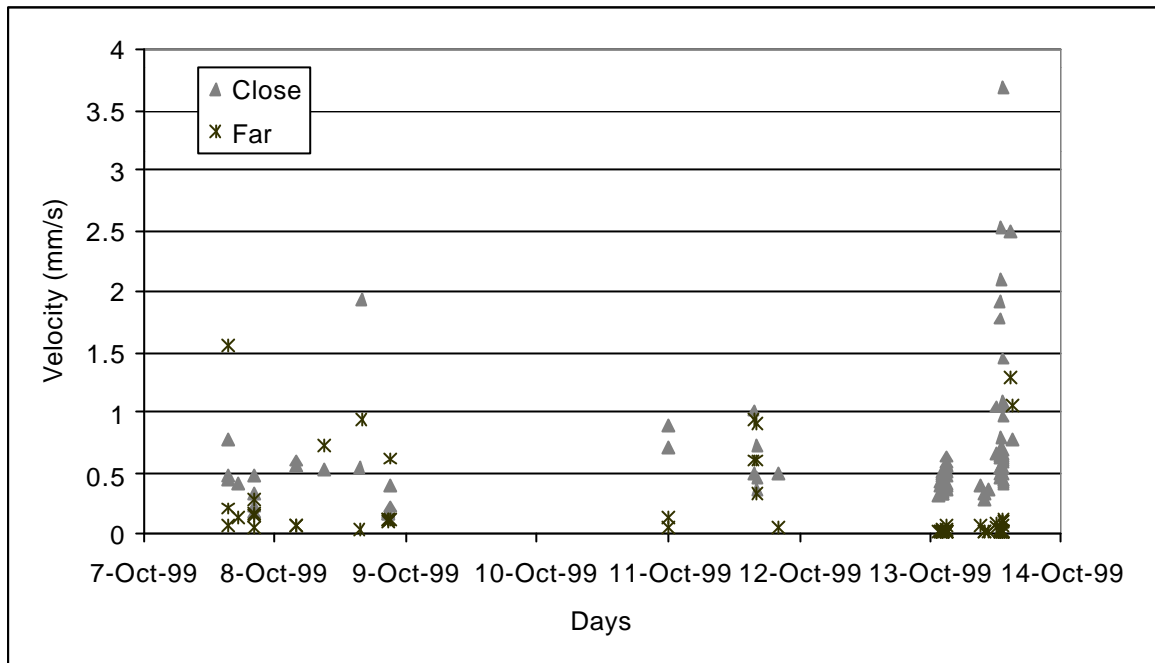


Figure 4.1.8 Peak particle velocities recorded in non-backfilled stope at Kopanang mine

Peak particle velocities recorded in the backfilled and non-backfilled stopes are compared to evaluate the capabilities of support to sustain the hangingwall stability under dynamic loading. Three factors are considered in the interpretation of the peak particle velocities: the source effect, effect of wave propagation through the rock, and the site effect. The source effect was eliminated as it was assumed that the same blasting charges were used in the backfilled and non-backfilled stope. To eliminate the effect of scattering and attenuation of the seismic waves during their propagation through the strata, the PPVs were corrected with a parameter, which involves multiplication by the square root of the distance to the face at the moment of registration. The results are plotted in Figure 4.1.9 The differences in the PPVs recorded in the backfilled and non-backfilled stopes are, therefore, related to the differences in the site effect, which reflects the differences in the existing support system. In the backfilled stope, the larger area of contact with the hangingwall and the stiffness of the backfill, very often similar to the stiffness of the fractured wall rock, have the effect of reducing the degree of movement of particles and hence their velocities. As a result, the backfilled stope hangingwall experienced much lower and less scattered PPVs compared to the non-backfilled stope hangingwall.

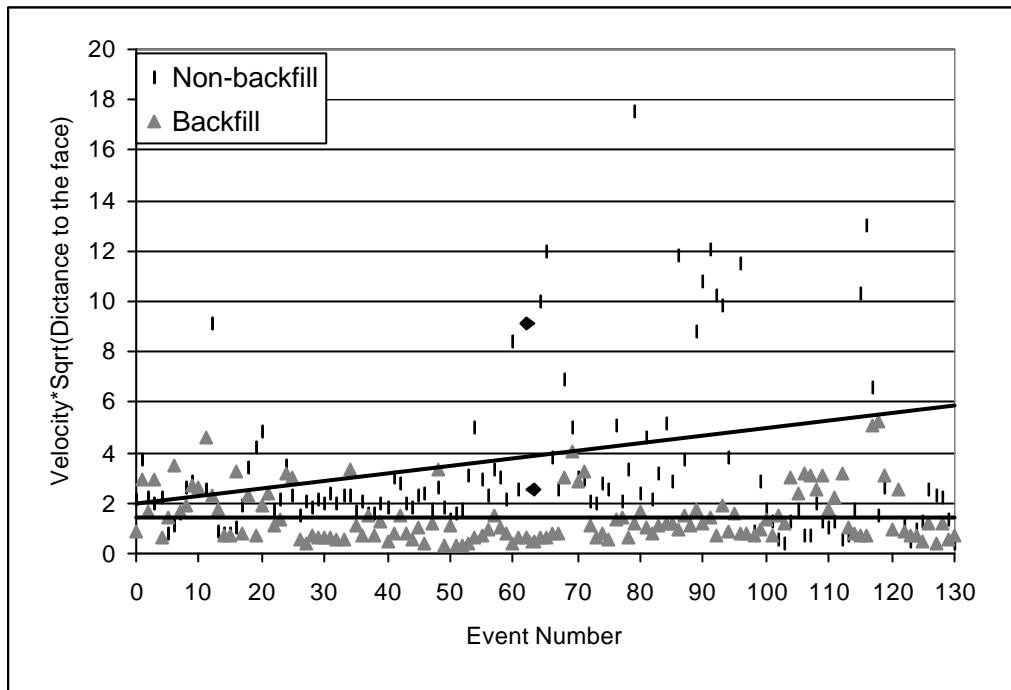


Figure 4.1.9 Peak particle velocities corrected for the distance from the face, recorded in backfill and non-backfilled stopes at Kopanang mine

The PPVs and crack deformations were studied during the seismic emission originating from two seismic regions. Source region 'A' was associated with the face of the panel where the instrumentation was installed and source region 'B' was associated with the face of an adjacent panel (Figure 4.1.10). The direction of quasi-static and dynamic deformations of the crack is also indicated in Figure 4.1.10.

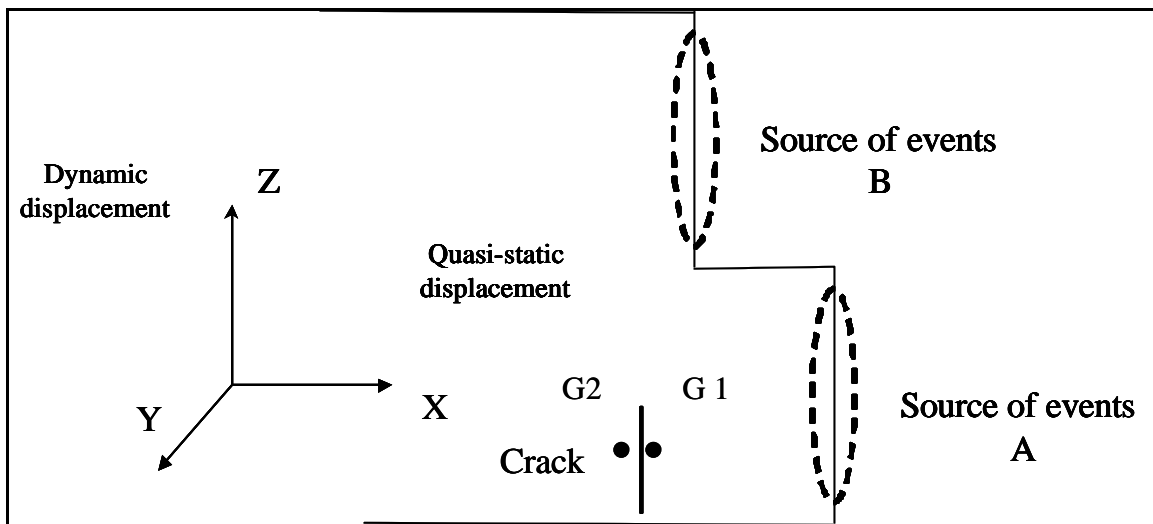


Figure 4.1.10 Schematic of seismic sources used in calculation of dynamic displacement for combined PPV and crack-gauge analysis

Two states of the crack were identified from the quasi-static measurements: State of relative opening and state of relative closing. The results are illustrated in Figure 4.1.11.

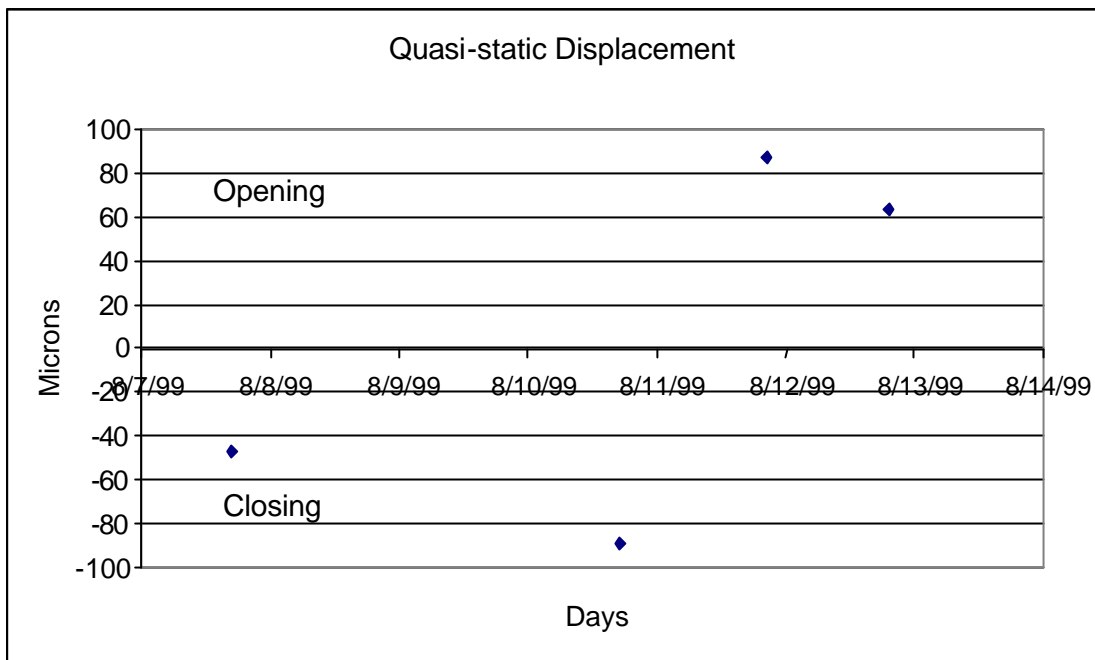


Figure 4.1.11 Movement of crack during period of observation

The dynamic displacement during the seismic event was also calculated and is presented in Figure 4.1.12. Seismic events from both source areas 'A' and 'B' (cf. Figure 4.1.10) were used in these calculations.

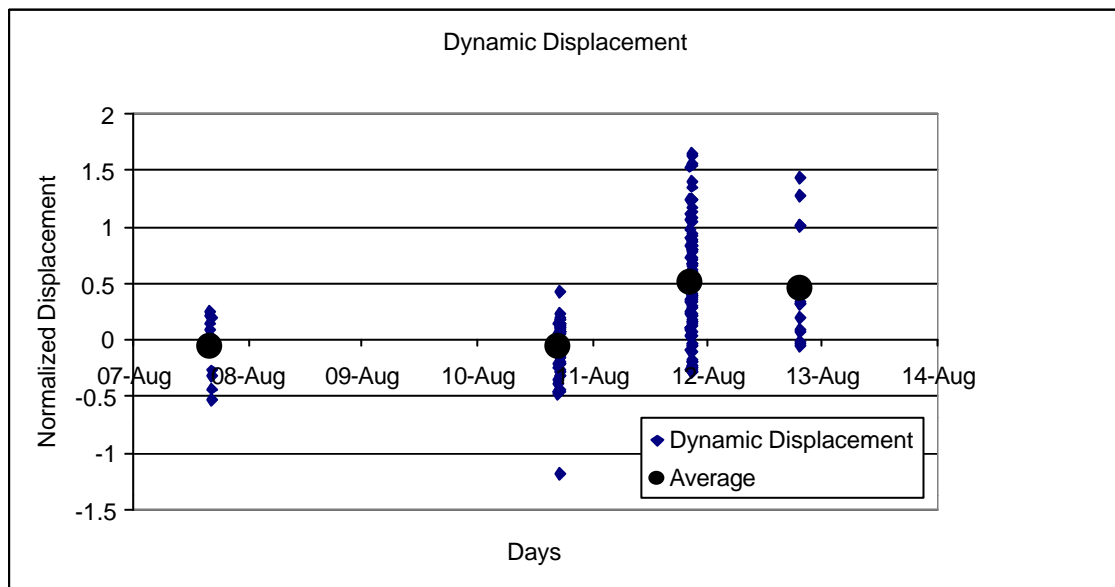


Figure 4.1.12 Dynamic displacement calculated using seismic events from source 'A' and source 'B'

It is indicated in Figure 4.1.12 that the state of relative opening is associated with the blasting operations at the monitored panel. Apparently, the stress transfer ahead of the face after the blasting leads to a relaxation of the stresses in the hangingwall and opening of the crack.

The state of relative closing of the crack was calculated during the seismic events generated from source 'B' located in the adjacent panel. However, it is not clear whether the state of relative closing is due to the reaching of equilibrium in the stress field around panel 'A' or the

influence of mining operations taking place in panel 'B'. The PPVs recorded during the states of 'relative opening' were about 3 times higher than the PPVs recorded during the states of 'relative closing'. However, the PPVs were all very low. Figure 4.1.13 shows the PPVs measured during each of these states.

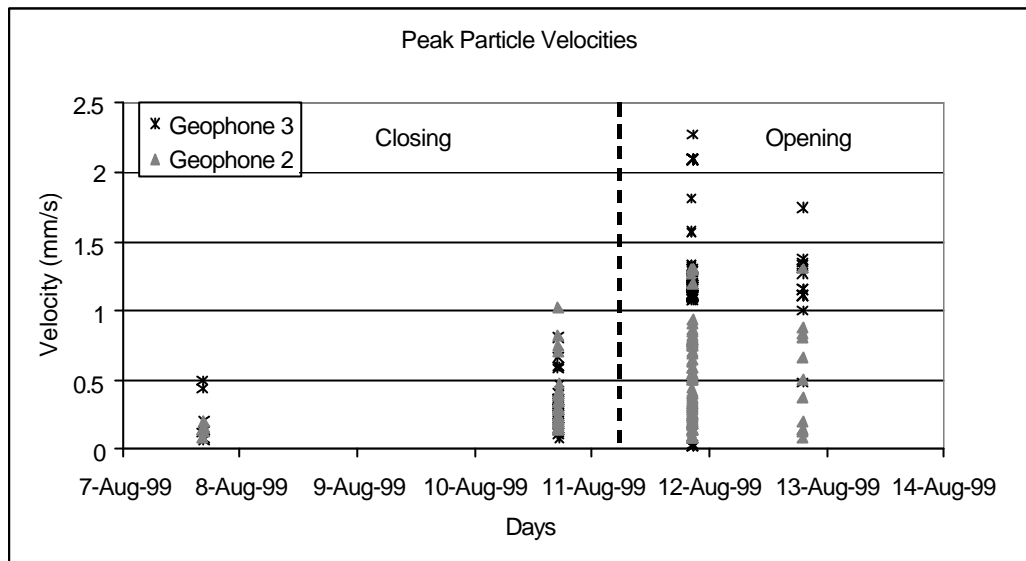


Figure 4.1.13 Peak particle velocities associated with crack state (i.e. opening or closing)

It is evident from Figure 4.1.13 that during the state of 'relative closing', the PPVs measured on both sides of the crack are similar but lower in value than the PPVs measured during the state of 'relative opening'. This result indicates that the crack deformations have a strong influence on the PPVs and probably also the hangingwall stability.

The findings can be summarised as:

- The rate of closure obtained in the non-backfilled site is higher than the rate of closure obtained in the backfilled site.
- The peak particle velocities measured in the non-backfilled stopes are higher and have larger dispersion than the peak particle velocities measured in the backfilled stopes.
- The peak particle velocities measured close to the face significantly exceeded the peak particle velocities measured in the back area in the backfilled site, whereas in the non-backfilled site this was not the case.
- In the earlier stage (close to the face), the backfill experienced a higher closure rate and PPVs than in the later stage (far from the face).
- During blasting time the closure rate increases rapidly.
- Crack deformations have a strong influence on PPVs and probably hangingwall stability: when the crack is opened, the PPVs are greater.

4.1.4 Numerical modelling

Figure 4.1.14 shows a plan view of the area of interest at Kopanang mine. Seven mining steps were considered from May 1999 to October 1999. The properties used for the model were Young's modulus of 50 GPa and Poisson's ratio of 0.2. The k-ratio was taken as 0.5.

With underground closure measurements, it was possible to calibrate the model parameters as well as the backfill parameters. Figure 4.1.15 compares the modelling results with underground measurements for both backfilled and non-backfilled sites. At the half span of 65 m, the backfill stresses reached the value of 13-15 MPa, which is well correlated with previous research findings.

Two different mining scenarios were considered. The first one is the actual mining scenario, with backfill, that was explained above. And the second scenario assumes the whole area as non-backfilled. The minimum principal stress in the hangingwall shows a significant difference between the two scenarios. The X-X' section through the stope (Figure 4.1.14) was taken in order to obtain the effects of backfill in the gully hangingwall. Backfill reduces by 25 per cent the area that is affected by tensile stress as well as the magnitude (Figure 4.1.16). When the half span is about 16 m, backfill starts increasing the horizontal stresses and this leads to better hangingwall conditions when backfill is used.

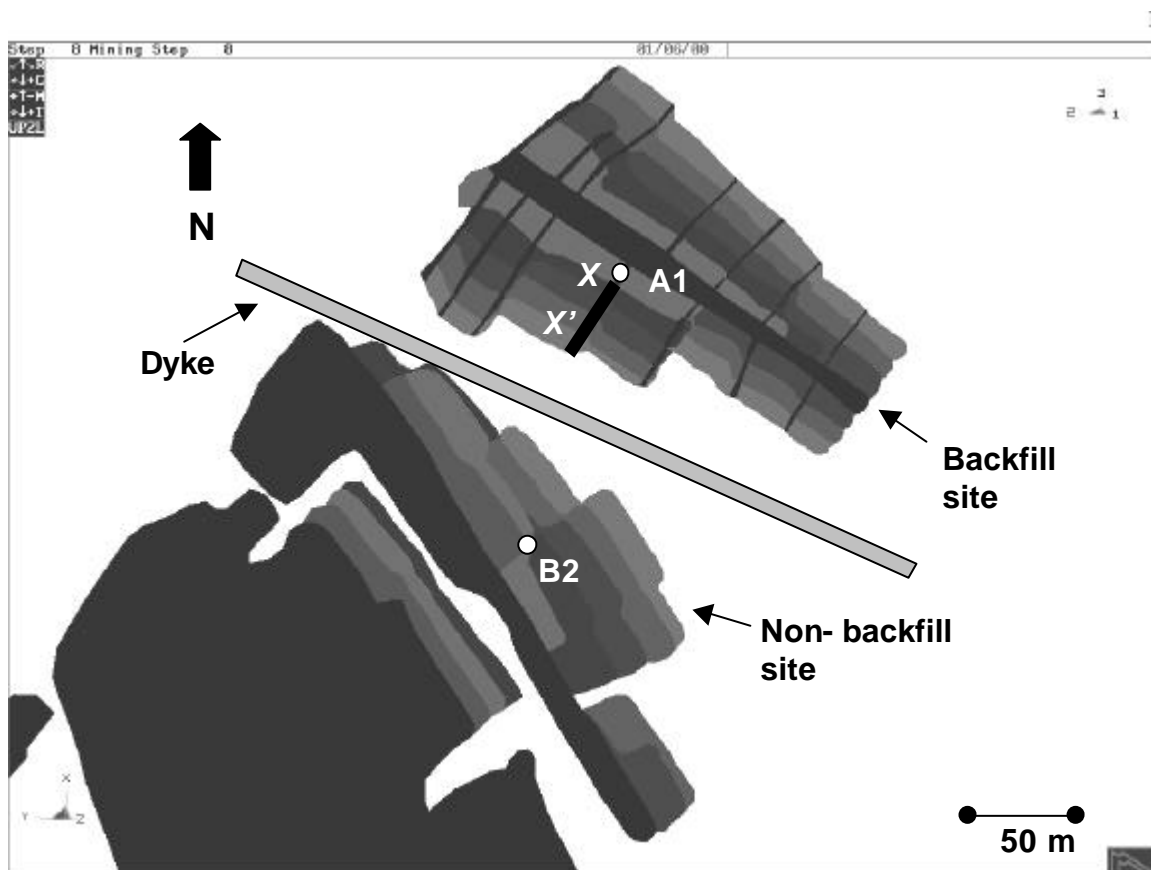


Figure 4.1.14 Modelled layout of Kopanang monitoring sites showing position of closure-ride stations A1 and B2

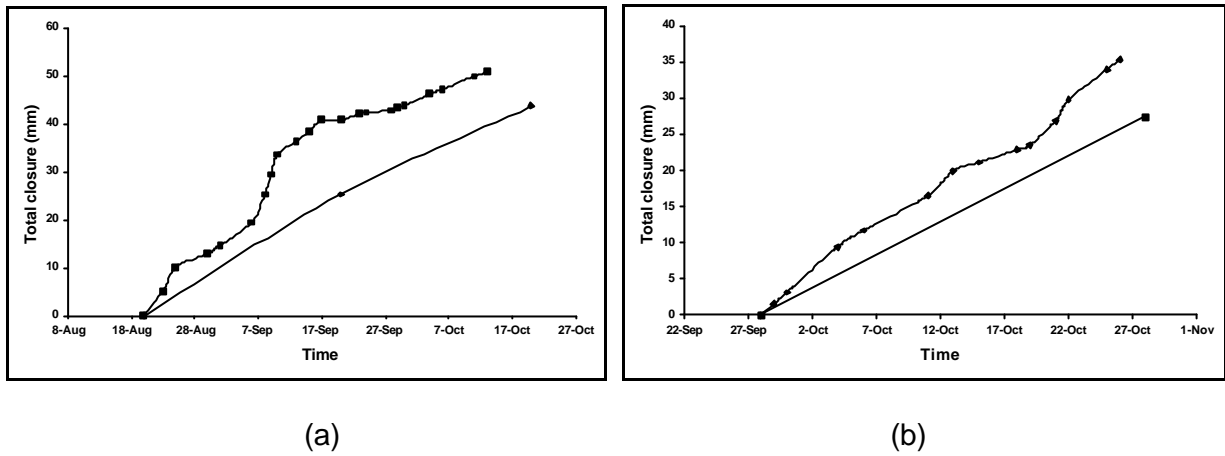


Figure 4.1.15 Comparison of underground total closure measurements and modelling results for (a) non-backfill site (at B2 in Figure 4.1.14) (b) backfilled site (at A1 in Figure 4.1.14) at Kopanang mine

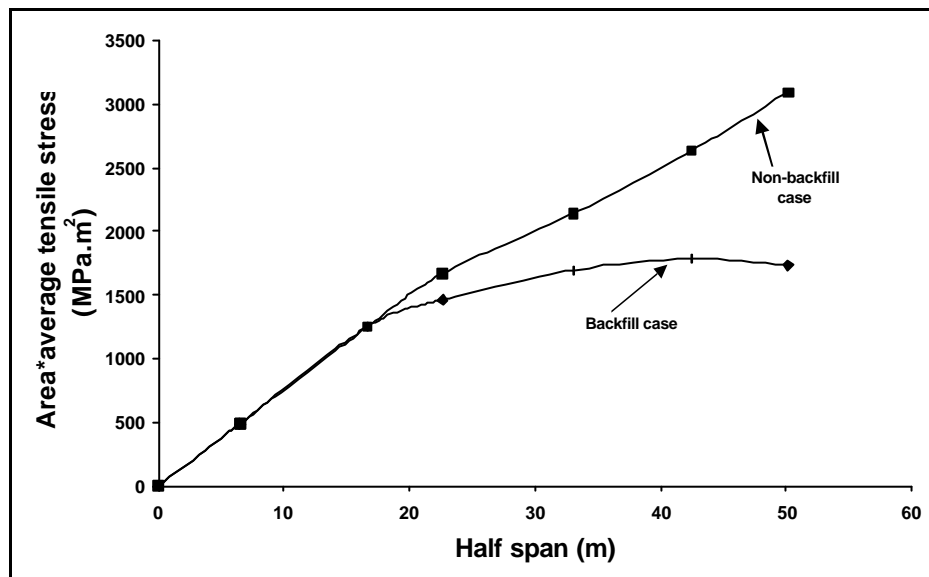


Figure 4.1.16 Modelled minor principal stresses at the hangingwall for backfill and non-backfill cases at Kopanang mine

4.1.5 Summary

The key findings from the Kopanang sites can be summarised as:

- Breaking of the argillaceous parting typically results in a rough hangingwall with mining-induced fractures visible.
- Irregular backfill placement results in gaps being left between bags, and such inconsistent placement may adversely affect strata conditions at the face.
- Generally, there is a larger variation in the dips of the fractures in backfilled panels (45° to 75°) than in conventionally supported panels (60° to 75°).
- Fracture spacing was found to be more variable in backfilled panels.
- Backfill reduces the magnitude of tensile stress as well as the area affected by it.

- The rate of closure obtained in the non-backfilled site is higher than the rate of closure obtained in the backfilled site.
- The peak particle velocities measured in the non-backfilled stopes are higher and have larger dispersion than the peak particle velocities measured in the backfilled stopes.
- Peak particle velocities close to the face in the backfilled site significantly exceed the peak particle velocities in the back area. This is not the case, however, for the non-backfilled site.
- Close to the face, backfill experiences a higher closure rate and PPVs than it does far from the face.
- Crack deformations have a strong influence on PPVs and probably hangingwall stability: when the crack is opened, the PPVs are greater.

4.2 Tau Tona mine

The monitoring programme at Tau Tona mine, Western Deep Levels, was initiated towards the end of 1999 in the 101/102 East Mini-longwall to monitor a typical 'Carbon Leader strike stabilising pillar with backfill' mining configuration at a depth of 3100 m. Three sites alongside the gullies of panels E1, E2 and E3 were monitored.

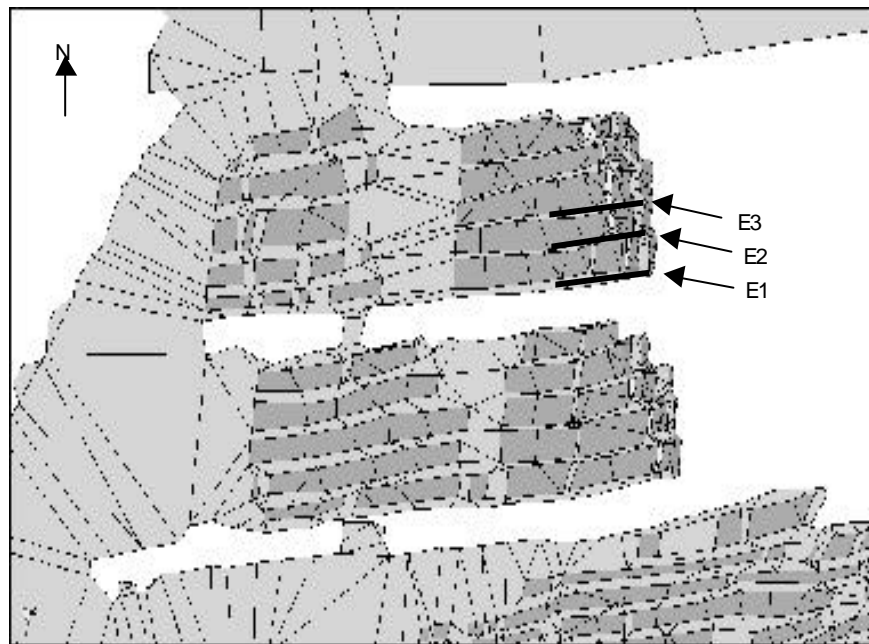


Figure 4.2.1 *Mining layout and position of monitoring instruments at Tau Tona 101/102 East Mini-longwall*

4.2.1 Review of installed support

Support at this site consists of conventional support components and backfill. Support quality and hangingwall condition were considered to be good.

4.2.1.1 Conventional support

Stope support consists of pre-stressed elongates, spaced 1.6 m on strike and 1.0 m on dip, together with backfill placed between these rows of elongates. Face area support is provided by the pre-stressed elongates which are installed at a maximum of 2.0 m from the face before the blast, hence there is no requirement for temporary support. Double composite packs are installed on the north side of the gullies and backfill is placed to within 1.0 m up-dip of these packs. There are no packs installed on the south sides of gullies. Instead, backfill is placed right up to the gully edge. The gully hangingwall is supported by rockbolts on a 1-2-1-diamond pattern with SplitSets in between.

4.2.1.2 Backfill

Backfill of a minimum porosity of 44 per cent is placed between the rows of pre-stressed elongates at a maximum of 4.5 m from the face before the blast. The coefficient of permeability of this backfill is 7.5×10^{-4} , which represents a tenth of the permeability of backfill achieved by other mines. The hyperbolic 'a' and 'b' parameters are 21.9 and 0.328, respectively. The material is well graded with a moderately low percentage of fines, with 6.1 per cent at -38 micron and 4.7 per cent at -10 micron. A reduction in the percentage of -38 micron might improve the drainage characteristics of the material. A summary of laboratory tests provides the following:

- It is evident from the laboratory analysis that Tau Tona is producing a well-graded backfill material with perhaps slightly too much -38 micron material.
- The amount of water in the backfill, after a few days, gives water-solids ratios of about 23 per cent. This is close to the minimum porosity level for these materials, or the level of porosity found at the bottom of the oedometer test stress-strain curve.
- The RD of the placed backfill appears to improve with time and indicates that it is achieving its objective of providing a stiff support within a few days.

4.2.2 Hangingwall condition

The hangingwall condition at this mine was, where possible, assessed by means of profiling and rock mass rating techniques.

4.2.2.1 Hangingwall profiling

The height of hangingwall steps profiling was carried out in the strike gullies of 102 E1 and 102 E3 panels of Tau Tona mine within 2 weeks of placement of backfill. The panels that these gullies serve were backfilled to within 3-4 m from the face as well as to the shoulder of the gullies.

Figure 4.2.2 and Figure 4.2.3 show the initial profile results obtained for the site at Tau Tona, panels E1 and E3 respectively, using the size of profile steps method of analysis (see section 3.2.1.1). The figures represent the profiles of the strike gullies at the time of placement of backfill. These results indicate an increased roughness, i.e. hangingwall fallout, at the E1 site (nearest to the strike pillar) compared to the E3 site (farthest from the strike pillar).

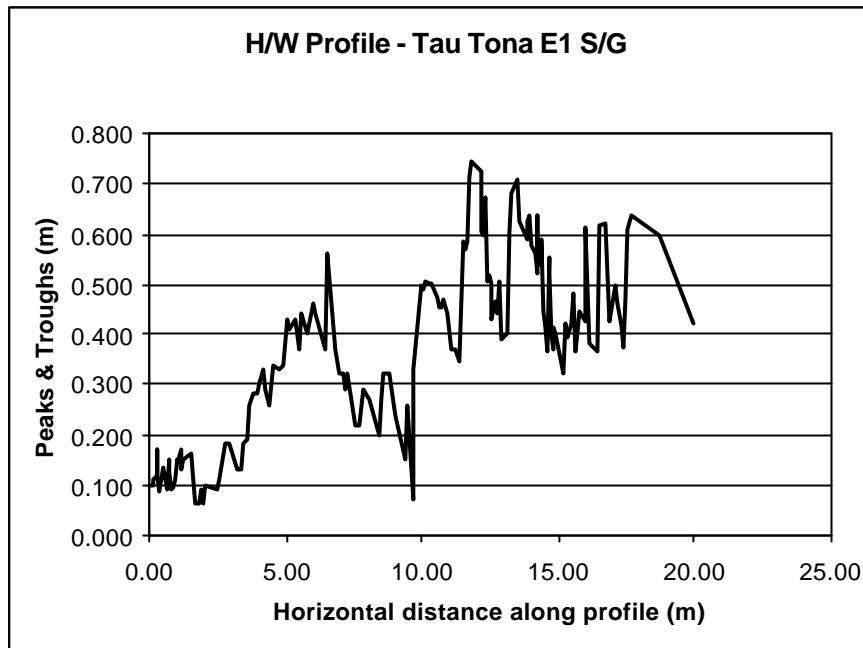


Figure 4.2.2 *Initial hangingwall profile in the strike gully of 102 E1 panel at Tau Tona mine*

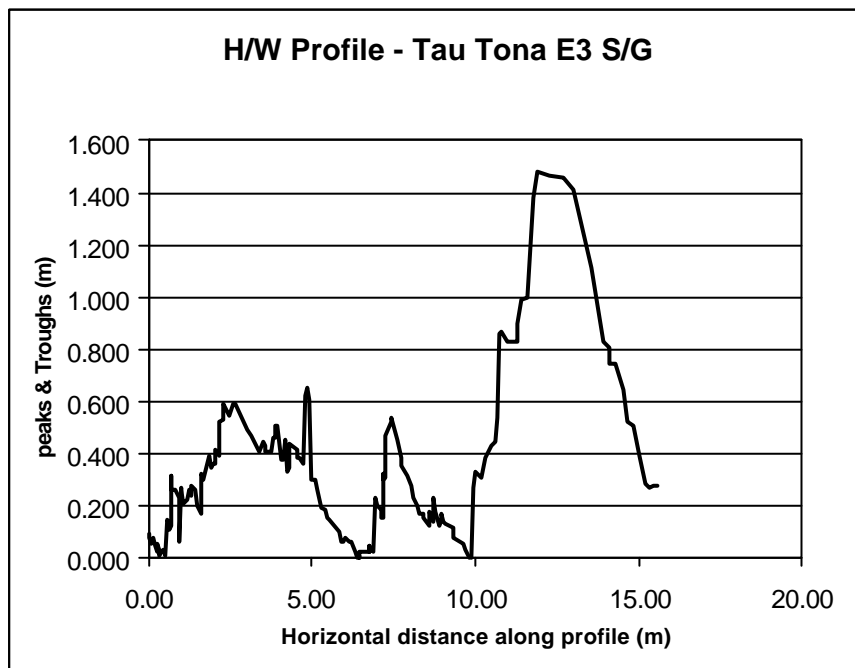


Figure 4.2.3 *Initial hangingwall profile in the strike gully of 102 E3 panel at Tau Tona mine*

Follow-up hangingwall profiles were carried out 2.5 months later in the E1 and E3 panels, at the same locations as used in the first profiles. A comparison of the two profiles for each gully (Figure 4.2.4 and Figure 4.2.5) shows sections over which fallout has occurred during the 2.5 month interval. During this interval significant fallout, and concomitant support installation, had occurred from the 8m mark onwards in panel E1 and 11 m onwards in panel E3, which prevented further measurements and comparison being made for these locations.

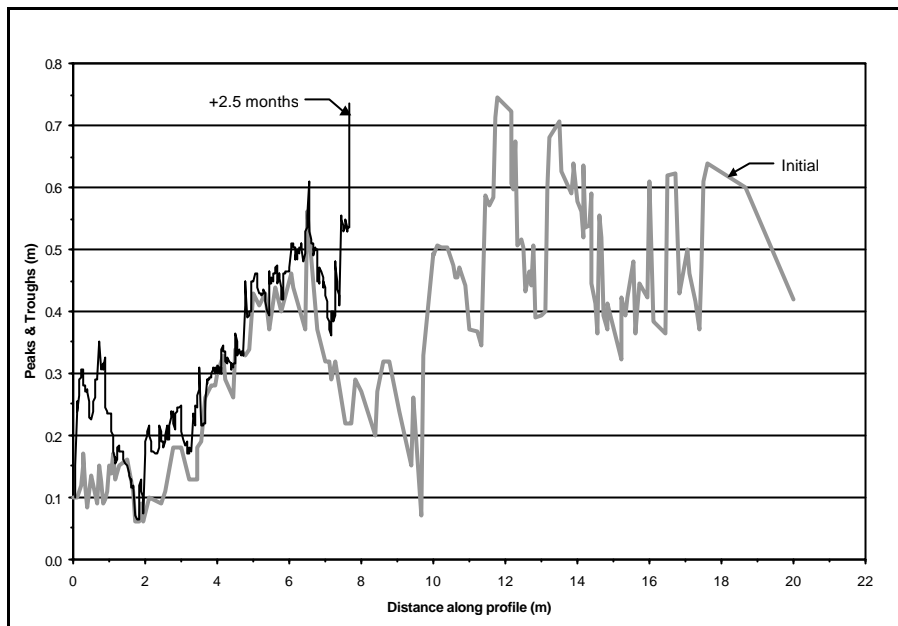


Figure 4.2.4 Comparison of initial hangingwall profile and follow-up hangingwall profile carried out 2.5 months later in 102 E1 panel at Tau Tona mine

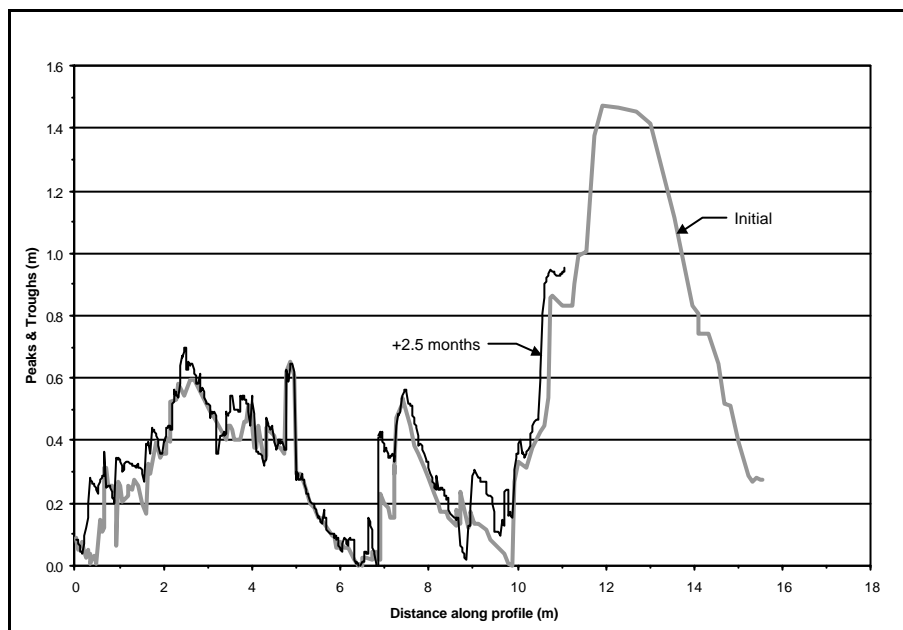


Figure 4.2.5 Comparison of initial hangingwall profile and follow-up hangingwall profile carried out 2.5 months later in 102 E3 panel at Tau Tona mine

In Table 4.2.1, the total length of the profiles and their average gradients are presented. The smaller these values are, the smoother the hangingwall. Table 4.2.1 shows computed values of total length of profile and gradient of hangingwall profiles made in the E1 and E3 panels. The initial values are compared with those obtained after 2.5 months to ascertain whether there has been a deterioration in hangingwall condition over time. As can be seen from Table 4.2.1, there was an increase in length of profile and average gradient in panel E1 from 21.8 m to 25.4 m and 68.6° to 75.1°, respectively. This indicates a slight deterioration in the gully hangingwall over 2.5 months. The same trend cannot be explained for the E3 panel where, although there was a slight increase of 1.4 m in total length of profile (indicating a slight deterioration after 2.5 months), the average gradient decreased slightly.

Table 4.2.1 Summary of hangingwall profile analysis for Tau Tona mine

Panel	E1		E3	
	Initial	+ 2.5 months	Initial	+ 2.5 months
Total profile length (m)	21.8	25.4	20.2	21.6
Average gradient (°)	68.6	75.1	85.7	84.3

Figure 4.2.6 shows hangingwall-parallel fractures (H) arching down to the siding face, i.e. pillar sidewall. These fractures were very numerous (e.g. 20 per metre) and created thin prisms in the hangingwall. Siding-parallel fractures were evident for 4m into the adjacent panel. These siding-parallel fractures curve such that in the second panel away from the pillar, steep dipping, stope-face parallel fractures are dominant (Figure 4.2.7).



Figure 4.2.6 Hangingwall-parallel fractures (H) arching down to siding face in 102 stope at Tau Tona mine



Figure 4.2.7 *Steep dipping, stope-face parallel fractures in second panel away from pillar in 102 stope at Tau Tona mine*

4.2.2.2 Rock mass rating

Measurements were made in the 102 E4 and 102 E3 gullies and in a dip gully located 3.5 m from the face in the 102 E3 panel (see Figure 4.2.8) for the purpose of conducting hangingwall rock mass ratings. Owing to operational difficulties, measurements were not possible next to the strike pillars in this stope and, therefore, a second stope (106 – below the instrumentation site) was also analysed (see Figure 4.2.8). However, measurements were not possible next to the strike pillars in this stope either and the analyses took place in the 106 E3 and 106 E2 gullies. A total of 10 windows were used in the assessment.

Table 4.2.2 shows the geotechnical properties of the geological discontinuities. Generally, only short lengths of the discontinuity surfaces were exposed and therefore the joint roughness measurements were performed on 0.1 m lengths. The measured offsets were corrected to the standard used in this report (0.5 m lengths) by using Barton's tables, thus enabling roughness comparisons to be made with other sites.

Four joint sets were identified and the orientations of these sets are shown in Figure 4.2.9. The strike direction of the fractures shown in Figure 4.2.10 follow the strike direction of joint set 3 (see Figure 4.2.9) rather than the strike of the face (see Figure 4.2.10).

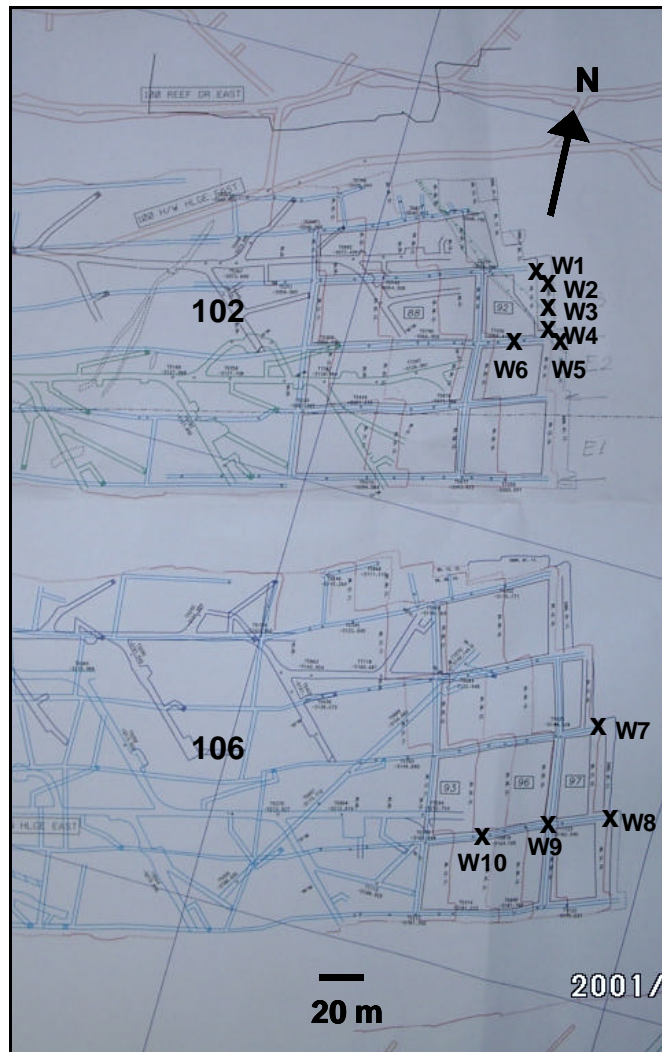


Figure 4.2.8 Plan showing the positions of survey windows at Tau Tona mine

Table 4.2.2 Discontinuity properties measured at the Tau Tona instrumentation site

Discontinuity type	Persistency	Roughness over 0.5 m	Infill type	Infill thickness	Rock type	Spacing
Set 1	>10 m	8 mm	Quartz	8 - 30 mm	Quartz	1.2 m
Set 2	>10 m	8 mm	Quartz & Cal	1 - 3 mm	Quartz	>5 m
Set 3	>10 m	8 mm	Cal	1 - 2 mm	Quartz	>5 m
Set 4	>10 m	8 mm	Quartz	8 mm	Quartz	>5 m
Fracture 1 & 2		8 mm	-	-	Quartz	cf. Figure 4.2.10

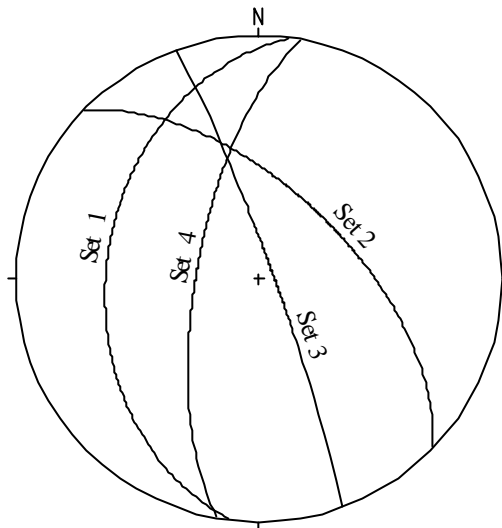


Figure 4.2.9 *Equal-area stereo net showing orientations of the four joint sets at Tau Tona mine*

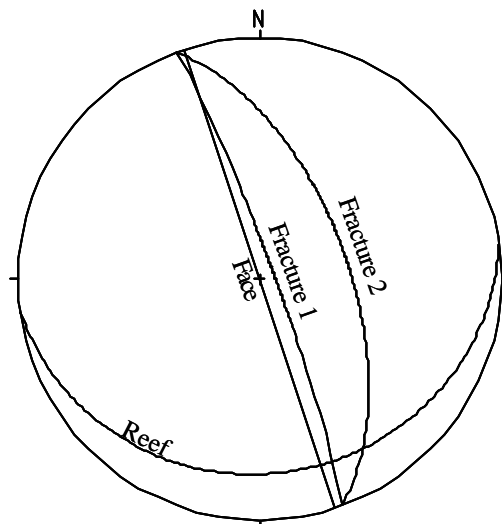


Figure 4.2.10 *Equal-area stereo net showing orientations of fracture sets observed at Tau Tona mine, compared to reef dip and face orientation*

4.2.2.3 Analyses performed in 102 stope

The results of the survey performed in the gully at the bottom of panel 102 E4 and in the dip gully located 3.5 m from the 102 E3 face are shown in Table 4.2.3, and those for the survey in panel 102 E3 are shown in Table 4.2.4. Only one window was possible in the 102 E4 gully. Three windows were analysed in the dip gully. The analysis in the dip gully was performed at right angles to the axis of the gully.

Table 4.2.3 Results of geotechnical survey in 102 E4 gully and 102 E3 dip gully at Tau Tona mine

Window number	Distance from face	Window length	RQD	RMR'	Q'	Average spacing of fractures	No. of shallow dipping fractures	No. of steeply dipping fractures
W1	10 m	10 m	18%	63	2.85	51 mm	48	148
W2	3.5 m	1.4 m	29%	68	4.59	34 mm	36	5
W3	3.5 m	1.4 m	21%	63	3.32	58 mm	7	17
W4	3.5 m	1.4 m	19%	63	3.01	37 mm	32	6

Table 4.2.4 Results of geotechnical survey in 102 E3 gully at Tau Tona mine

Window number	Distance from face	Window length	RQD	RMR'	Q'	Average spacing of fractures	No. of shallow dipping fractures	No. of steeply dipping fractures
W5	2 m	5 m	21%	63	3.32	32 mm	76	15
W6	28 m	5 m	23%	63	3.64	57 mm	59	28

4.2.2.4 Analyses performed in 106 stope

The results of the surveys performed in the gullies at the bottom of panels 106 E3 and 106 E2 are shown in Table 4.2.5 and Table 4.2.6 respectively. One and three windows were analysed in the E3 and E2 gullies respectively.

Table 4.2.5 Results of geotechnical survey in 106 E3 gully at Tau Tona mine

Window number	Distance from face	Window length	RQD	RMR'	Q'	Average spacing of fractures	No. of shallow dipping fractures	No. of steeply dipping fractures
W7	10 m	5 m	18%	66	2.85	104 mm	37	11

Table 4.2.6 Results of geotechnical survey in 106 E2 gully at Tau Tona mine

Window number	Distance from face	Window length	RQD	RMR'	Q'	Average spacing of fractures	No. of shallow dipping fractures	No. of steeply dipping fractures
W8	4 m	10 m	16%	66	2.53	110 mm	78	13
W9	40 m	10 m	16%	66	2.53	83 mm	68	52
W10	75 m	5 m	14%	66	2.22	111 mm	35	10

4.2.2.5 Discussion

Trends associated with time deterioration could not be established. The spread of results in the windows located at the same distance from the face as shown in Table 4.2.3 was as great as in the windows located far from the face, as shown in Table 4.2.6. This strengthens the argument that there is little or no increase in the fracture intensity with time.

4.2.3 Ground motion and closure

The scope of the ground motion monitoring programme was to investigate the variations in PPVs and the amount of closure in the areas strongly influenced by the stabilising pillar (e.g. close to the pillar) compared to areas less influenced by the pillar (e.g. in the middle of the longwall). The mining layout and position of the monitoring sites are shown in Figure 4.2.1.

Seismic data recorded by the mine's seismic network was also collected for the equivalent area and time period covered under the monitoring programme. A hypocentral map of the seismic events recorded by the mine network during the period of observation is shown in Figure 4.2.11. It can be seen from the Figure 4.2.11 that most of the seismic events are associated with the face advance. It is also interesting to note that some of the large events in the area are associated with the pillars.

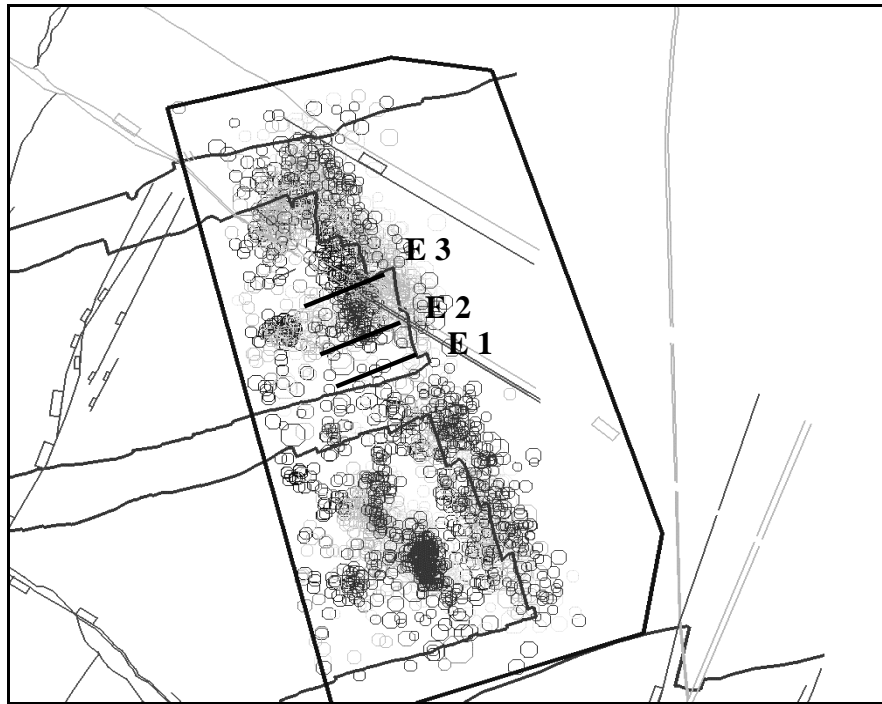


Figure 4.2.11 A hypocentral map of the seismic events located in section 336 during the period of observation. Data obtained from Rock Mechanics Department, at Tau Tona mine

In order to distinguish the effect of the pillar on local support requirements, PPVs generated by small microseismic events associated with the face advance are compared for different distances from the pillar. In this way the effect of the events located on the pillar and the large events located on geological structures, e.g. dykes and faults, is eliminated. The PPVs measured close to the face in the E1, E2 and E3 panels are plotted in Figure 4.2.12.

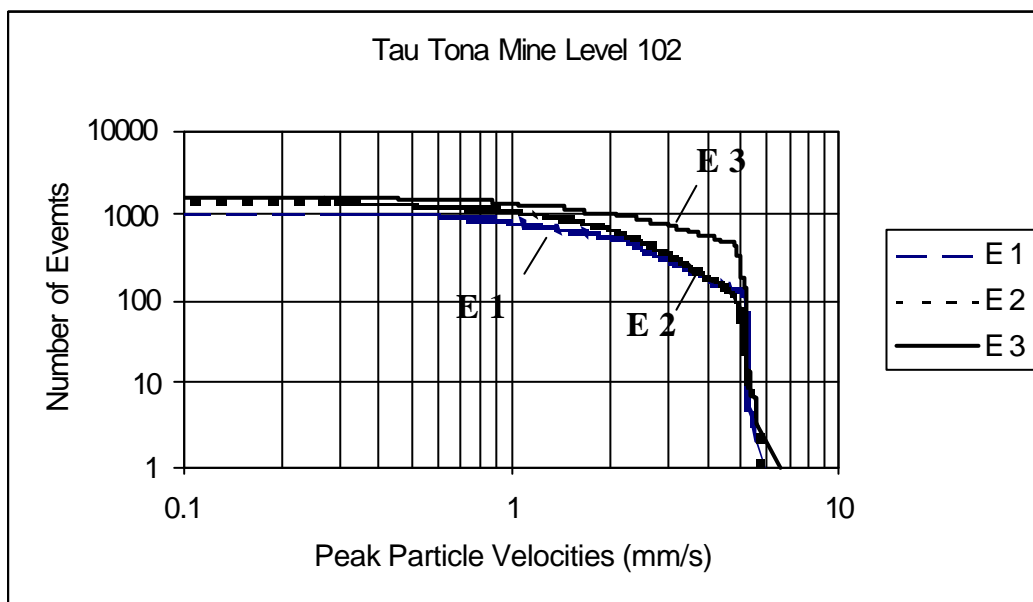


Figure 4.2.12 Peak particle velocities recorded in E1, E2 and E3 panels at Tau Tona mine

It is evident from Figure 4.2.12 that the number of seismic events with PPVs between 1 mm/s and 5 mm/s increases with increasing perpendicular distance from the pillar, i.e. the number of these events and the associated PPVs are greater in E3 panel than E1 and E2 panels. However, the differences between E1 and E2 are not as significant. This result illustrates the effect of pillar-associated seismicity having an affect on the closest panels, e.g. E1 panel. The effect of this pillar seismicity is more evident in Figure 4.2.13, were additional strong seismic events associated with face advance are included.

To include the events with the larger PPVs, a previously created procedure (Spottiswoode, 1997) was modified and incorporated as a standard feature in software for the automatic derivation of PPVs from saturated seismograms. A plot of the 'corrected' PPVs is shown in Figure 4.2.13. The number of events is normalised for the period of observation.

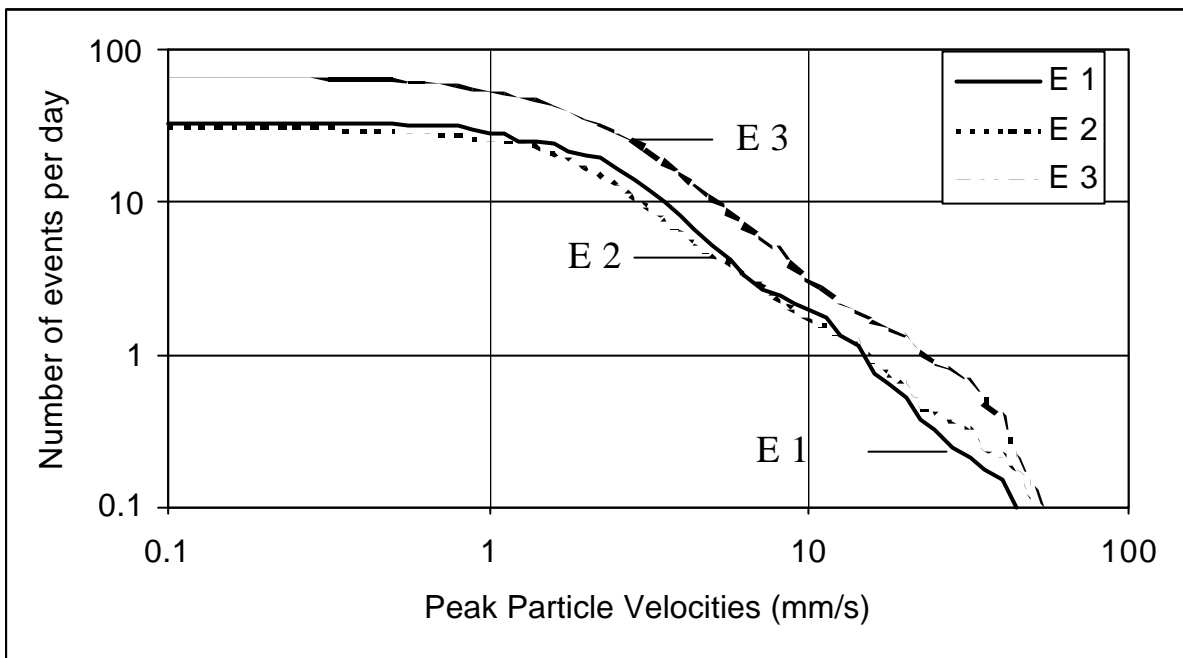


Figure 4.2.13 Peak particle velocities corrected for saturated trace over entire period of observations for E1, E2 and E3 panels at Tau Tona mine

All curves in Figure 4.2.13 have similar slopes, indicating a similar seismicity characteristic. However, the number of events recorded in the E3 panel is greater than the number of events recorded in E1 and E2 panels. Interestingly, the PPVs recorded in E1 panel are higher than the PPVs recorded in E2 panel. Even though this difference is not pronounced it does indicate that with increasing face advance the seismicity associated with the pillar adjacent to a given monitoring position also increases. This behaviour conforms to the changes in Average Pillar Stress (APS) with respect to the face advance shown in Figure 4.2.14 (York, 1997).

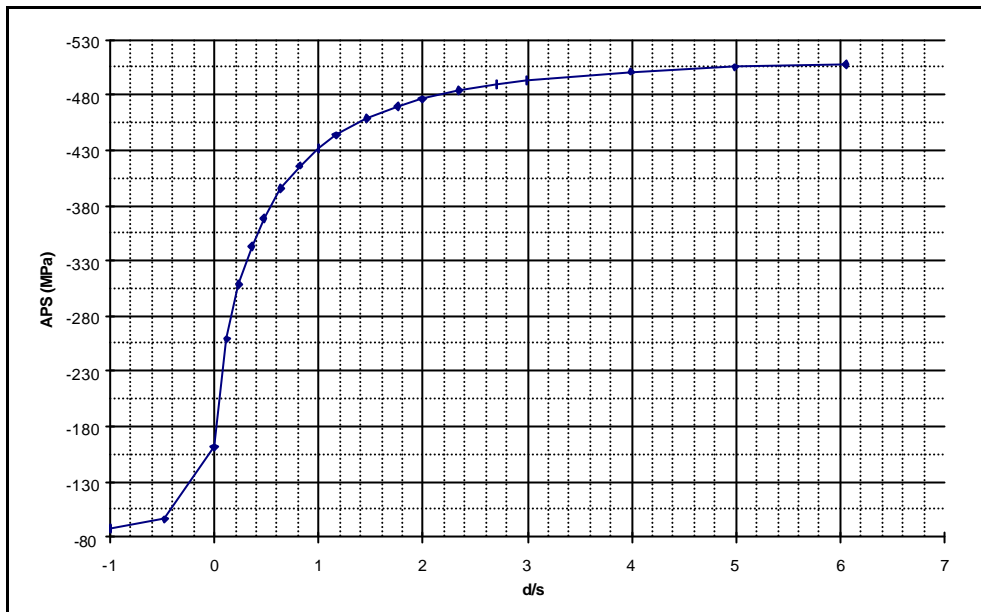


Figure 4.2.14 Average Pillar Stress (APS) along a pillar as a function of distance to the face (d) normalised for the stope span (s), determined from MINSIM-D modelling (York, 1997)

Additional expansion of the dynamic range to record the strong ground movements was done for each configuration by over damping one of the geophones. The PPVs recorded at E1 and E3 panels using this channel are shown in Figure 4.2.15.

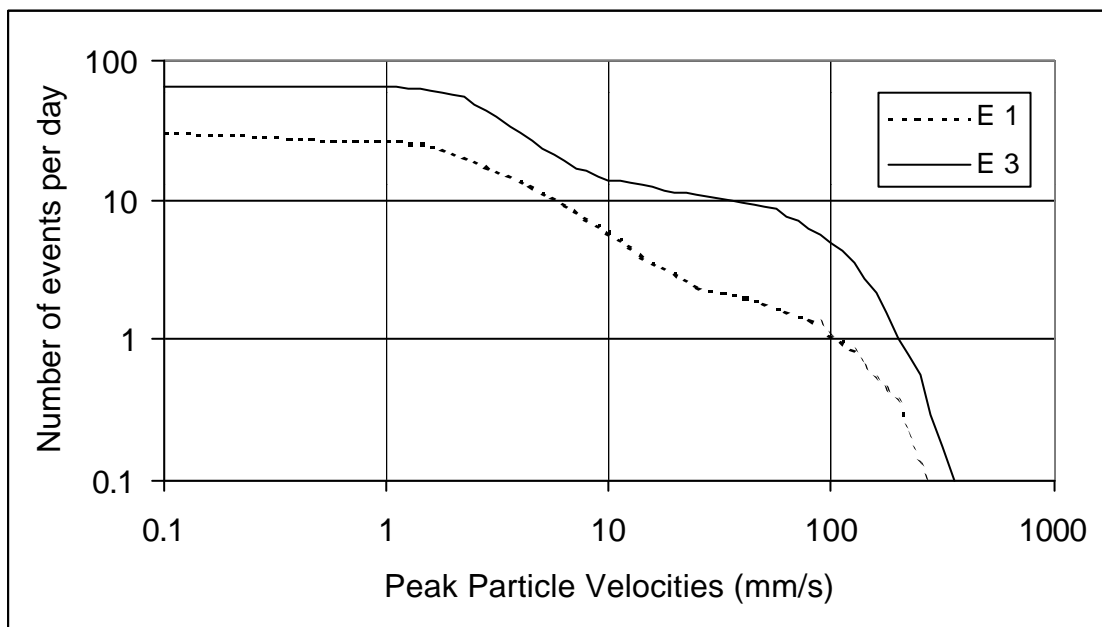


Figure 4.2.15 Peak particle velocities recorded by a super over damped channel especially designed to handle large PPVs at Tau Tona mine

The trend of an increasing number of events and their PPVs with increasing distance from the pillar is also shown by the results in Figure 4.2.15.

The effect of backfill on the PPVs is illustrated by comparing PPVs measured close to the face, where the backfill is still soft, to PPVs measured far from the face, where the backfill is stiffer and provides better support. The PPVs recorded in E1, E2 and E3 panels are shown in Figure 4.2.16, Figure 4.2.17 and Figure 4.2.18 respectively.

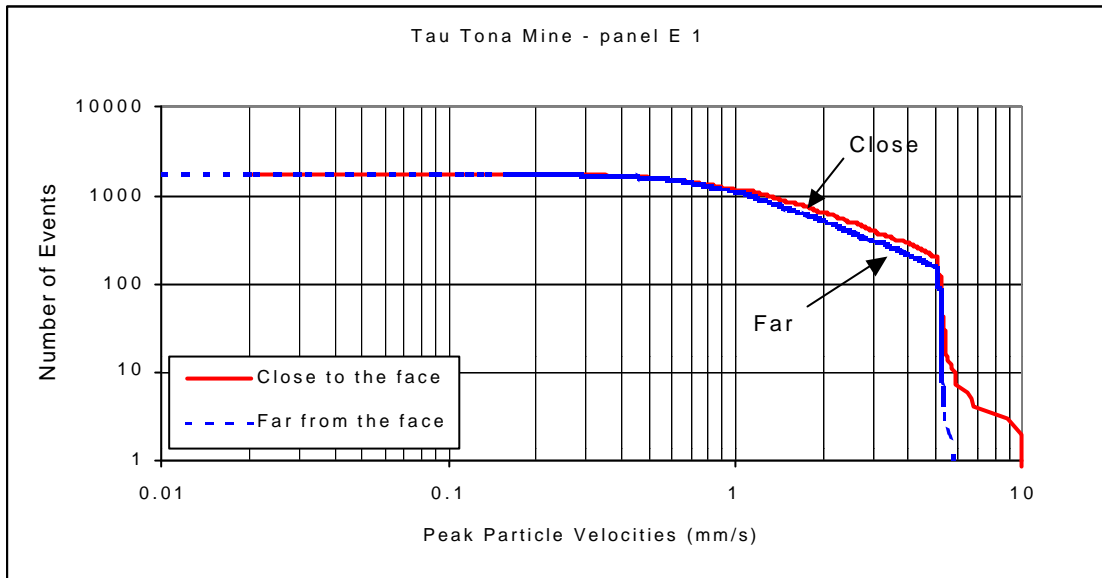


Figure 4.2.16 Peak particle velocities recorded close to the face (upper line) and far from the face (lower line) in E1 panel at Tau Tona mine

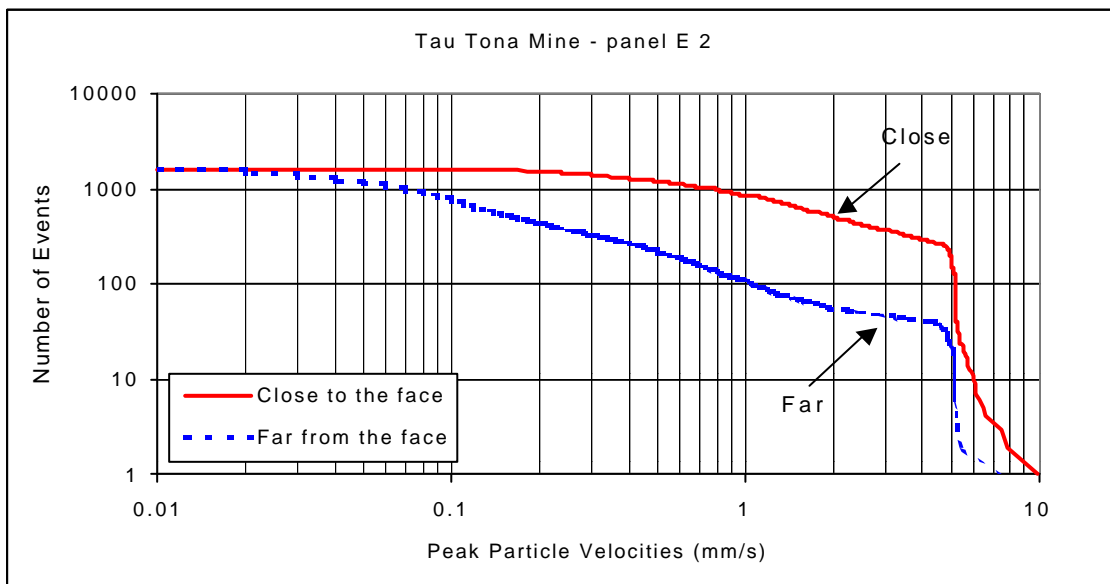


Figure 4.2.17 Peak particle velocities recorded close to the face (upper line) and far from the face (lower line) in E2 panel at Tau Tona mine

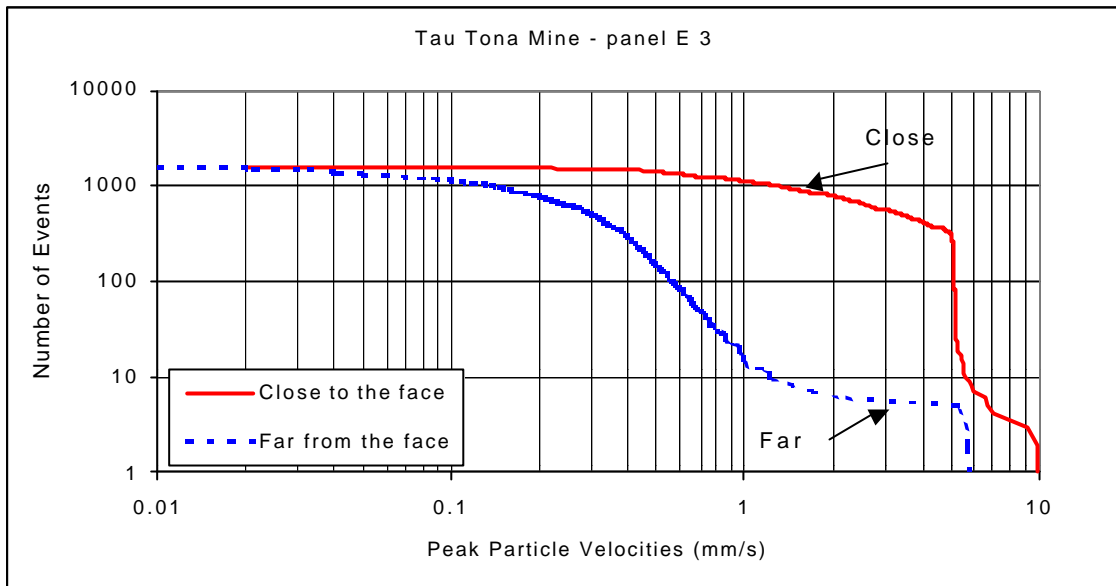


Figure 4.2.18 Peak particle velocities recorded close to the face (upper line) and far from the face (lower line) in E3 panel at Tau Tona mine

It can be seen from Figure 4.2.16, Figure 4.2.17 and Figure 4.2.18, that the PPVs measured close to the face are higher than the PPVs measured far from the face. The effect of the seismicity induced by an increase in APS (cf Figure 4.2.14) is evident for E1 panel (Figure 4.2.16), where the difference between PPVs close to the face and far from the face is smaller than the corresponding differences obtained for E2 and E3 panels.

The rate of closure measured in E1 panel is shown in Figure 4.2.19. The measurements were taken during the first 10 days after the installation at two points: close to the face and far from the face. The closure rate measured in a similar configuration in E3 panel is shown in Figure 4.2.20.

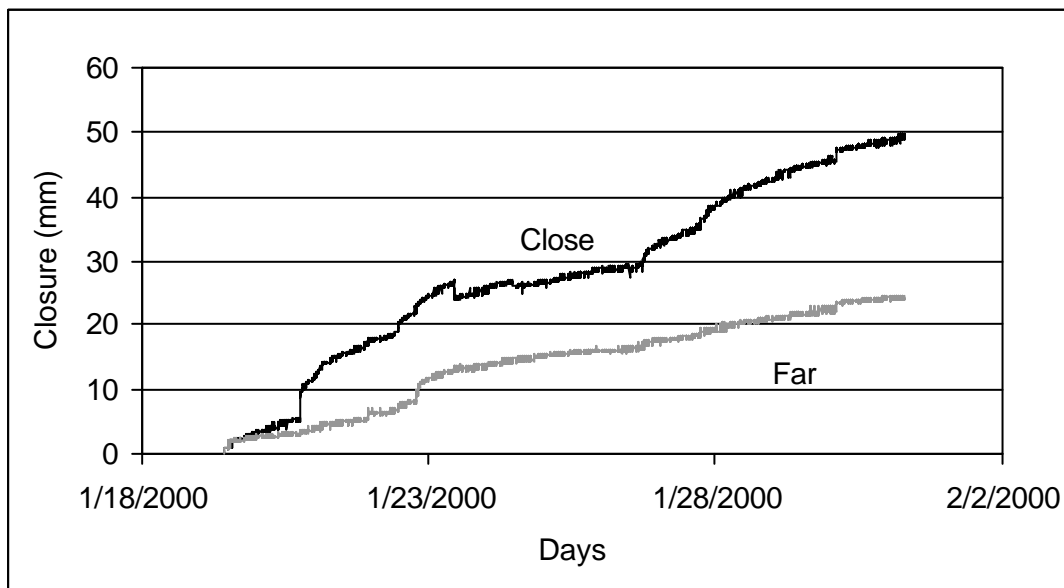


Figure 4.2.19 Closure rate recorded in 102-E1 panel close to the face and far from the face at Tau Tona mine

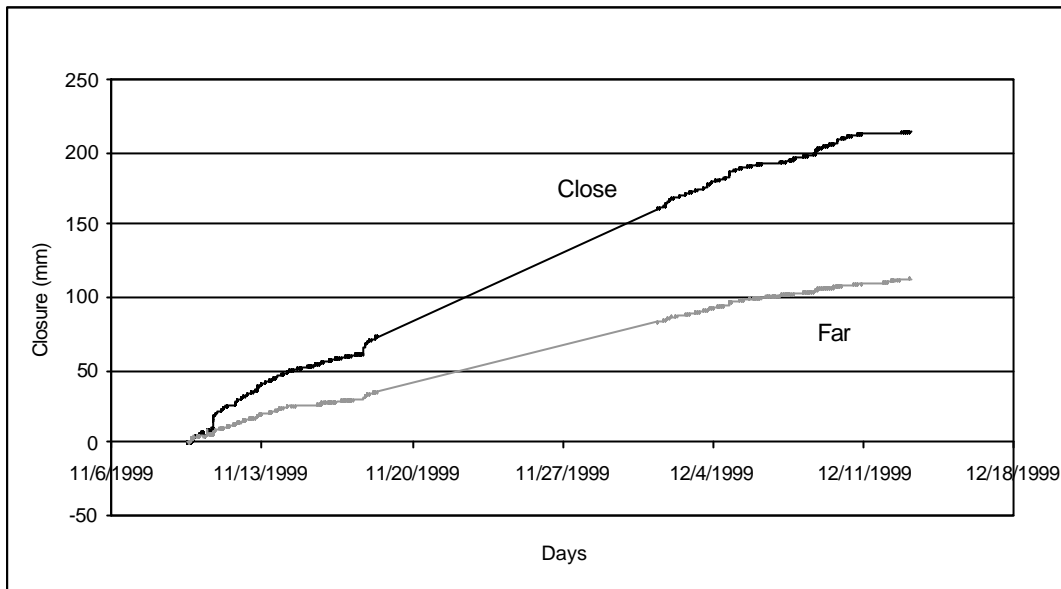


Figure 4.2.20 Closure rate recorded in 102-E3 panel close to the face and far from the face at Tau Tona mine

It can be seen from Figure 4.2.19 and Figure 4.2.20 that the amount of closure increases with increasing distance from the pillar for both points of measurement, close to the face and far from the face. The rate of closure close to the face is higher than the rate of closure far from the face for both E1 and E3 panels.

Additional processing of seismic data recorded by the mine seismic network was carried out in order to estimate the support interaction in conditions of strong seismic loading. The strong seismic events ($M > 0.5$) located in section 336 were extracted from the mine's seismic data base (obtained from Rock Mechanics Department, Tau Tona Mine) and correlated with the seismic events recorded in E1, E2 and E3 panels. Thirty events were recorded in all data sets. The frequency-magnitude distribution for these events is shown in Figure 4.2.21.

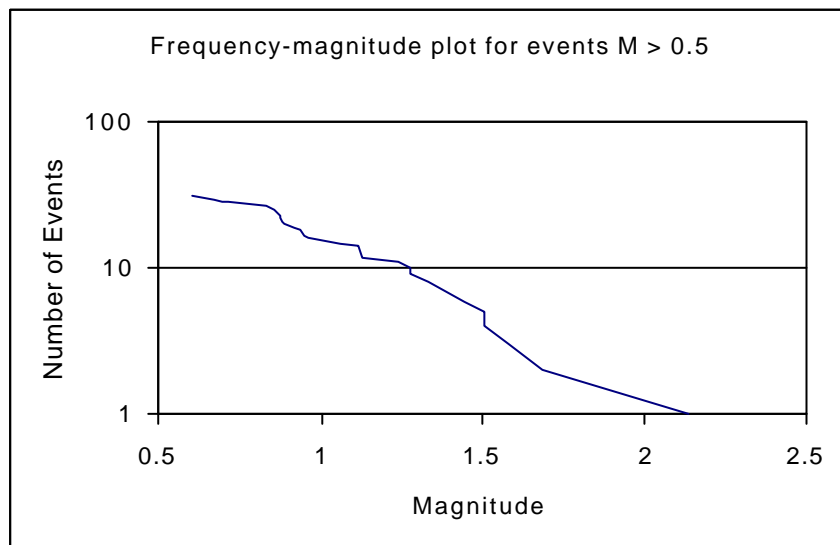


Figure 4.2.21 Frequency-magnitude distribution for seismic events recorded by the mine network and the recording configurations installed in E1, E2 and E3 panels

The PPVs recorded in E1, E2 and E3 panels for these same events were compensated for the attenuation with the hypocentral distance of each event, are shown in Figure 4.2.22.

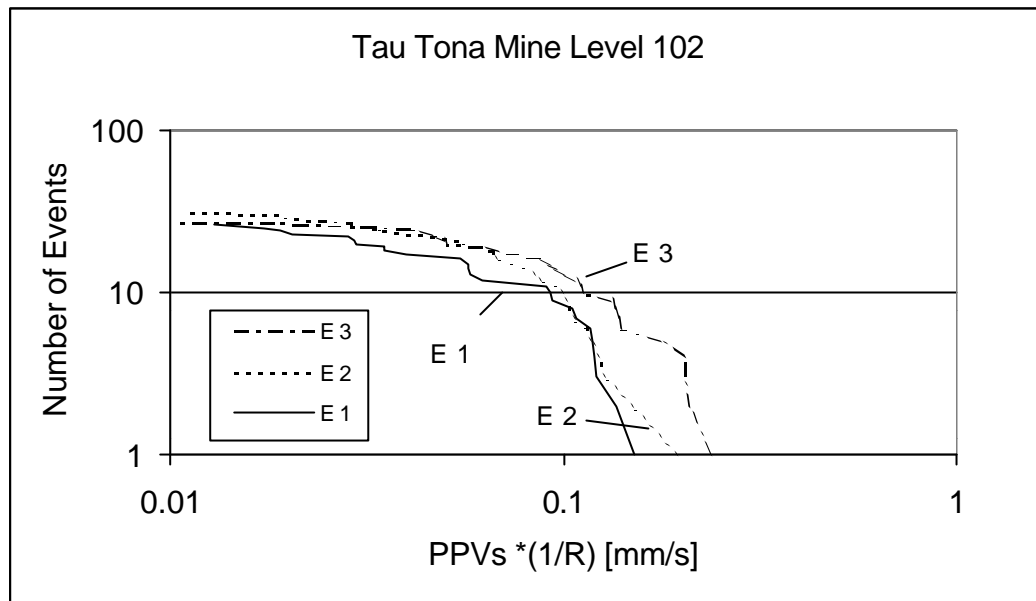


Figure 4.2.22 Peak particle velocities from 28 strong seismic events recorded in Level 102, E1, E2 and E3 panels; $(1/R)$ is the attenuation factor of the PPVs with the hypocentral distance

It is evident from Figure 4.2.22 that the PPVs generated by the strong events located in the area of interest increase with increasing perpendicular distance from the pillar, i.e. from E1 panel towards E3 panel (which is closer to the centre of the stope). The effect of the increasing macro seismicity associated with the face advance (illustrated in Figure 4.2.16) is not visible here as a special over-damped channel was used to record the strong seismic events.

During a seismic event, the support is loaded by dynamic force. The support behaviour during this loading can be characterised by the following equation (Wagner, 1984):

$$D = Cv^2$$

where: D is the dynamic closure defined as the difference in convergence at the beginning and at the end of a seismic event calculated for each panel and each seismic event;

v is the peak particle velocity estimate for each event; and

C is a constant representing the specific site and support conditions

The dynamic closure and support behaviour during seismic events are analysed for this site. The peak particle velocities are compared to the dynamic closure for each panel. The theoretical relationship of these two parameters is also calculated. In the calculation of theoretical dynamic displacement, corrections for the energy absorption criteria, suggested by Roberts (1999), were applied. Both the theoretical and observed results are shown in Figure 4.2.23, Figure 4.2.24 and Figure 4.2.25, respectively.

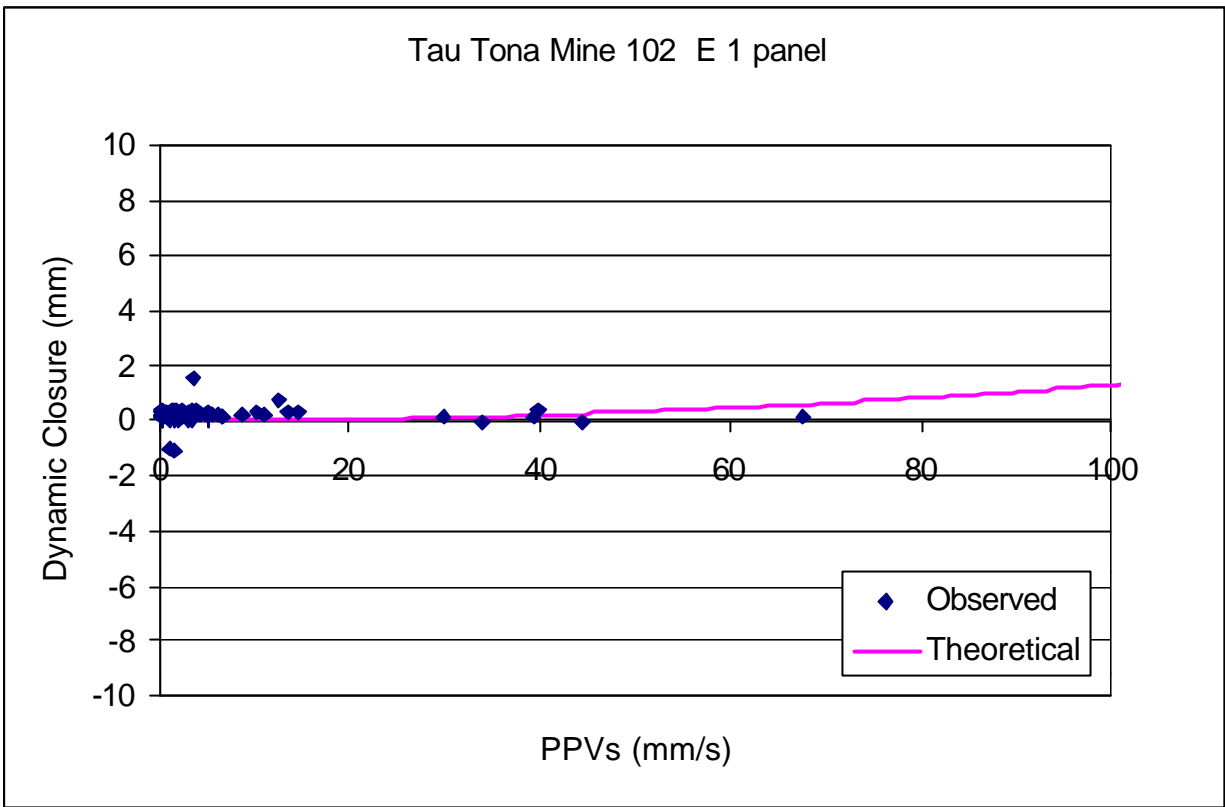


Figure 4.2.23 Dynamic closure as a function of PPV obtained for E1 panel at Tau Tona mine

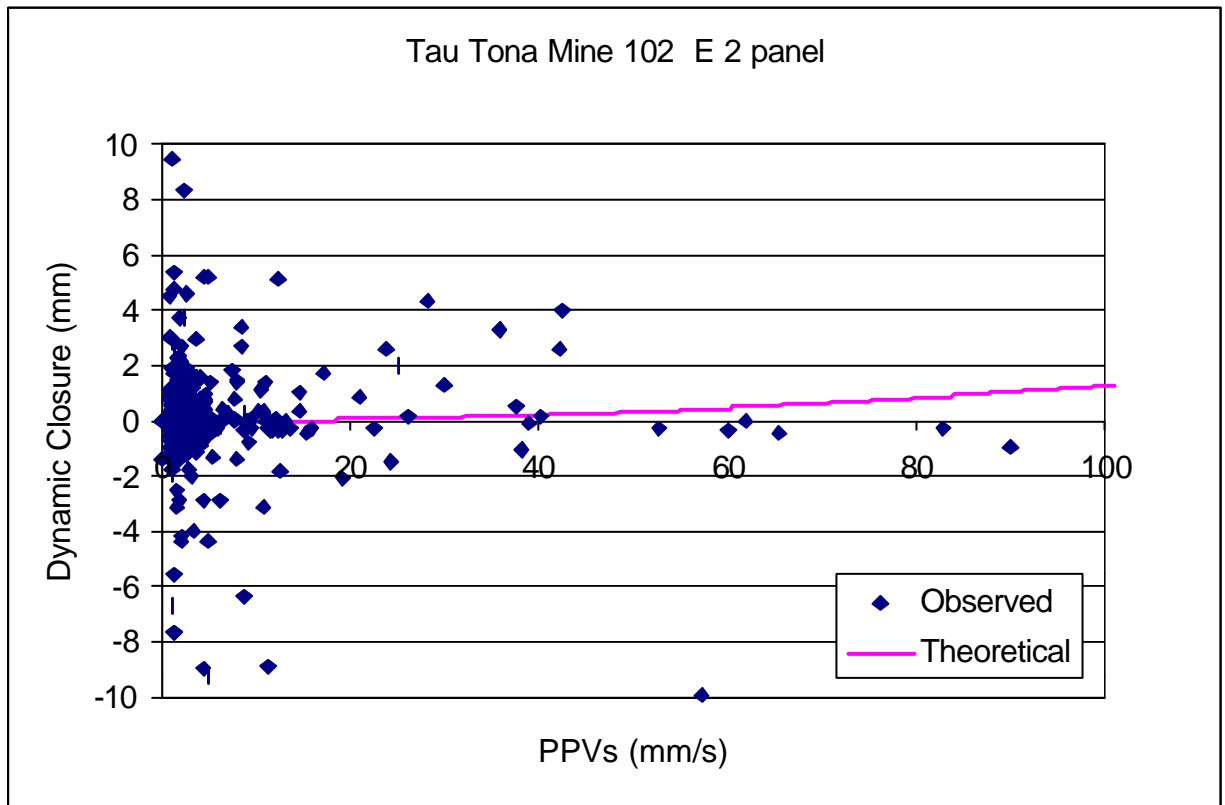


Figure 4.2.24 Dynamic closure as a function of PPV obtained for E2 panel at Tau Tona mine

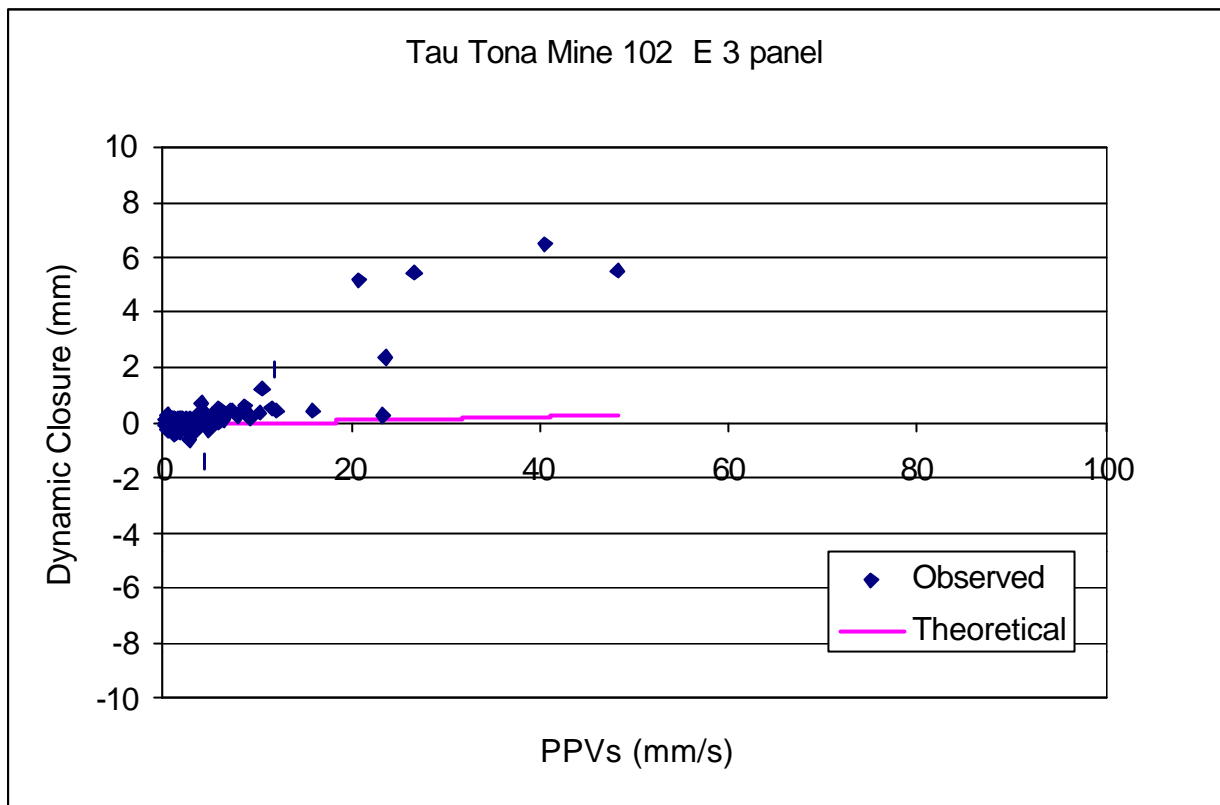


Figure 4.2.25 *Dynamic closure as a function of PPV obtained for E3 panel at Tau Tona mine*

A study of Figure 4.2.23, Figure 4.2.24 and Figure 4.2.25 shows that dynamic closure in the area close to the pillar, panel E1, is less affected by the PPVs. The area away from the pillar, i.e. panels E2 and E3, shows an increase in dynamic closure with an increase of PPVs. This effect is used further as an indicator for high and low resistance support systems.

4.2.4 Numerical modelling

Figure 4.2.26 and Figure 4.2.27 show the vertical stress modelled along a transect line in the strike stabilising pillar adjacent to the monitoring site at Tau Tona mine as a function of monthly face advance, for backfilled and non-backfilled scenarios respectively. The effect of the backfill is reflected as an almost 50 per cent reduction in pillar stress. For comparative purposes, Figure 4.2.28 summarises the initial and final vertical stress profiles along the stabilising pillar for backfill and non-backfill cases. Figure 4.2.29 shows the modelled build-up of vertical stress with increasing face advance at a fixed position in the pillar adjacent to a corresponding monitoring position, for backfilled and non-backfilled cases.

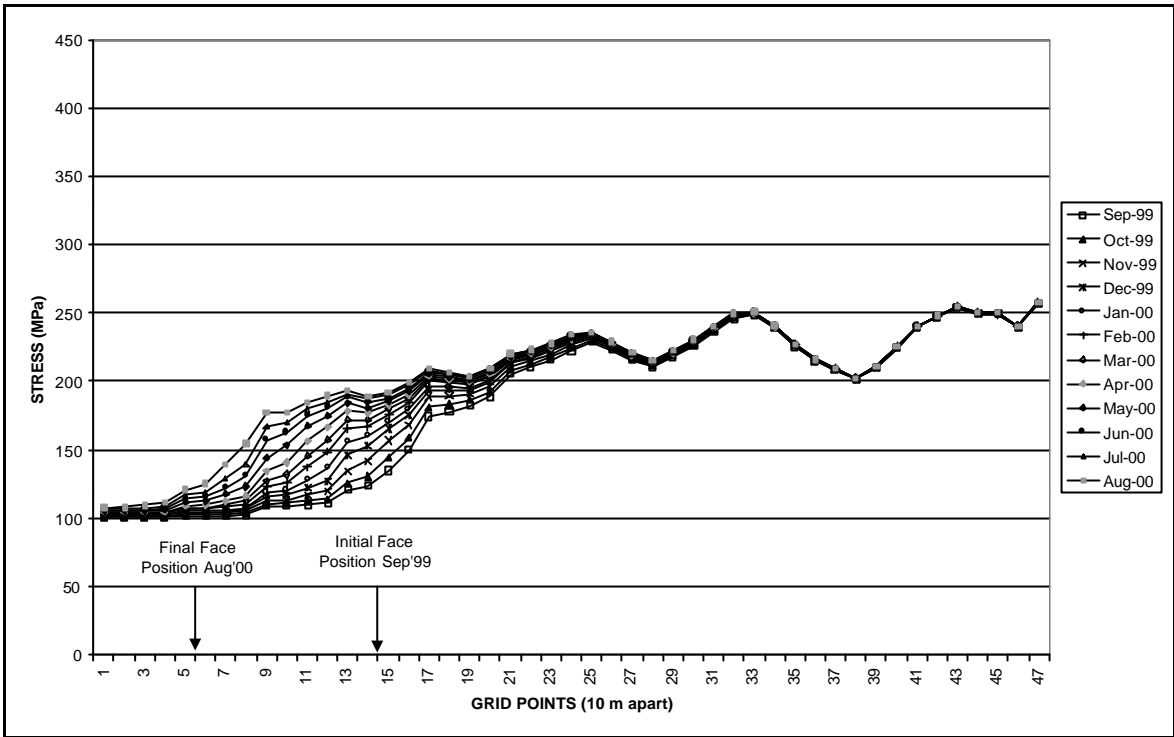


Figure 4.2.26 Modelled vertical stress profiles along strike stabilising pillar with backfill for given monthly face position at Tau Tona mine

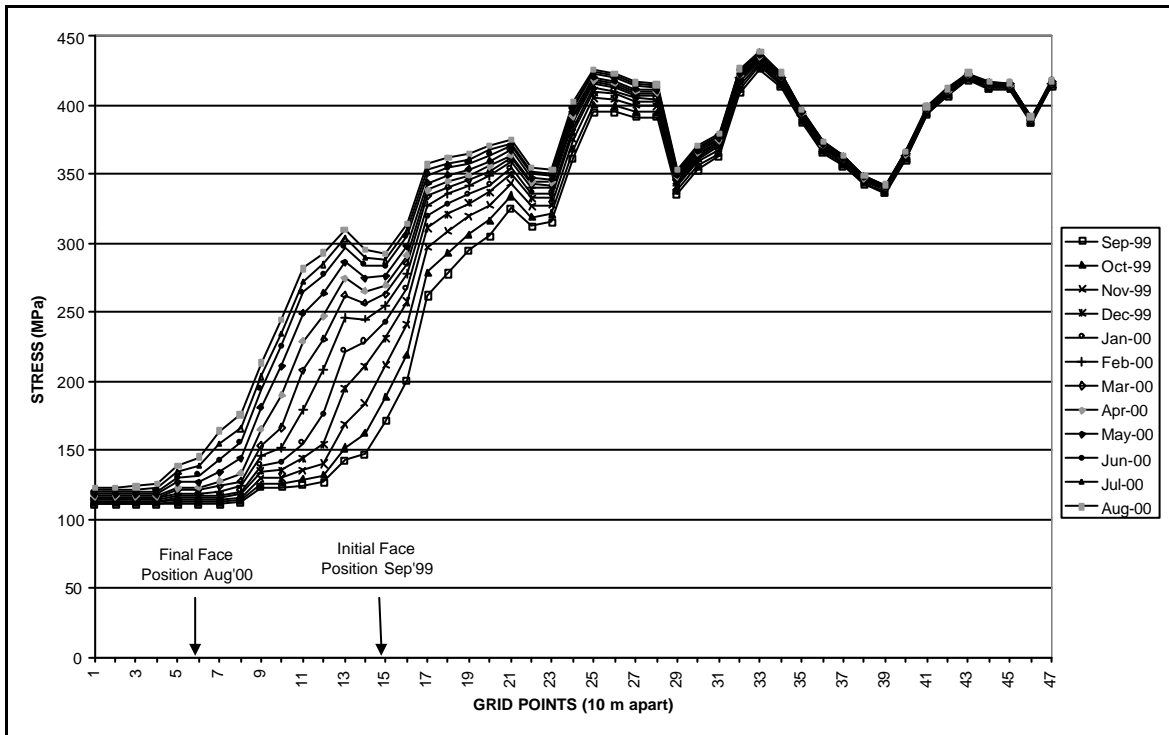


Figure 4.2.27 Modelled vertical stress profiles along strike stabilising pillar without backfill for given monthly face position at Tau Tona mine

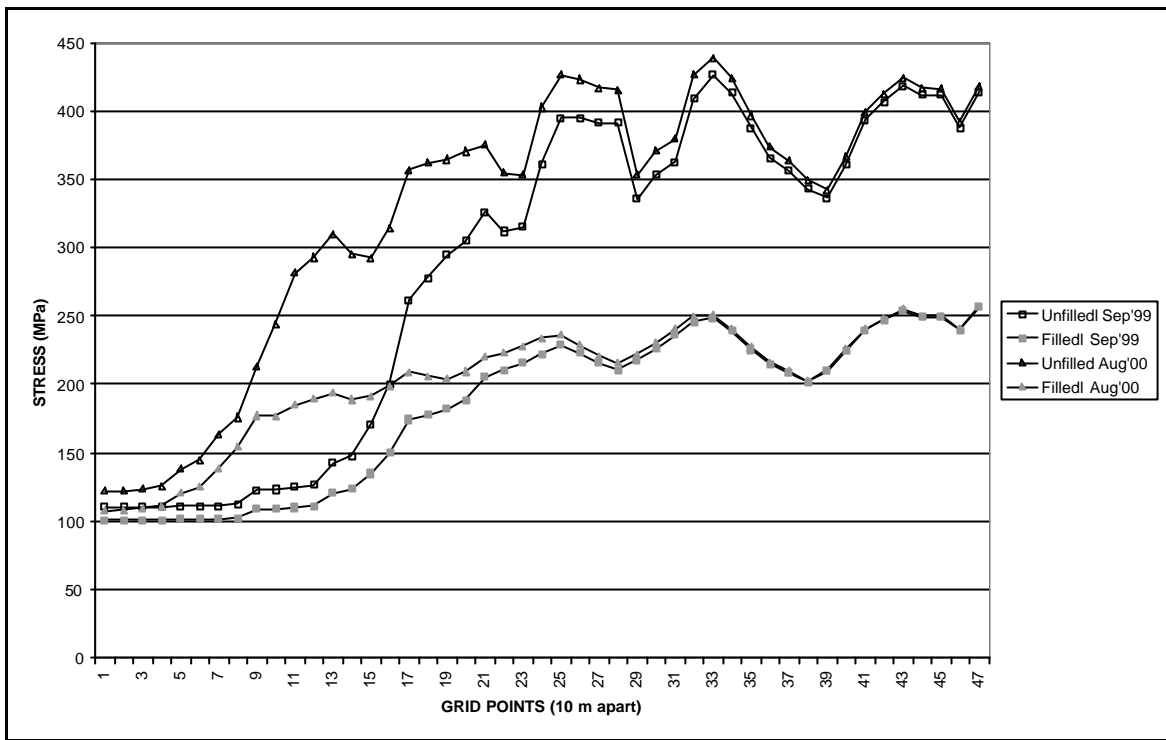


Figure 4.2.28 Comparison of initial and final modelled vertical pillar stress profiles with and without backfill at Tau Tona mine

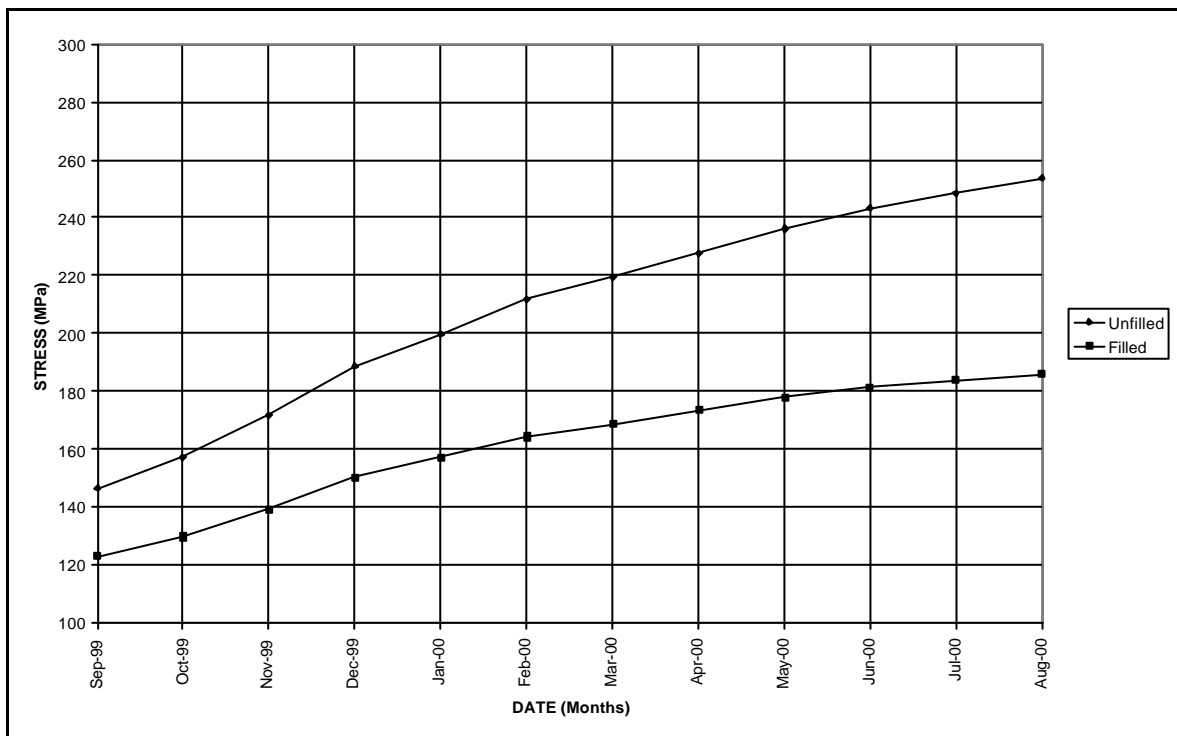


Figure 4.2.29 Modelled vertical pillar stress at a fixed location as a function of monthly face advance with and without backfill at Tau Tona mine

The graph in Figure 4.2.29 shows the difference in the increase in vertical pillar stresses and the absolute values with and without backfill over a period of 11 months for an advancing face. This result is in a good agreement with the increase in the micro seismic activity associated with the edge of the pillar as determined through the ground motion (PPV) study (section 4.2.2.2).

4.2.5 Summary

The key findings from the Tau Tona site can be summarised as:

- An increased roughness, i.e. hangingwall fallout, is indicated nearest to the strike pillar compared to a position more distant from the strike pillar.
- A slight deterioration in hangingwall condition is noted over time in the gully adjacent to the strike stabilising pillar. The same situation is, however, not evident for gullies more distant from the strike pillar.
- There is little or no increase in fracture intensity in gully hangingwalls with time.
- PPVs measured close to the face are higher than the PPVs measured far from the face.
- The seismicity associated with the stabilising pillar increases with increasing face advance, for a given position adjacent to the pillar.
- As perpendicular distance from the stabilising pillar increases there is an increase in PPVs generated by the microseismic events associated with face advance.
- PPVs generated by all seismic events increase with perpendicular distance from the pillar. However, there is an increase in PPVs in the panel adjacent to the pillar due to an increase in average pillar stress, which is associated with face advance.
- There is an increase in the amount of closure with increasing distance from the stabilising pillar, in cases close to the face and far from the face.
- The rate of closure close to the face is higher than the rate of closure far from the face.
- Dynamic closure close to the pillar is less affected by PPVs. While further away from the pillar there is an increase in dynamic closure with an increase in PPVs.
- Numerical modelling indicates that backfill reduces strike pillar stress by almost 50 per cent at this site.

4.3 Deelkraal Mine

The monitoring of a 'VCR strike stabilising pillar with backfill' mining layout was undertaken at Deelkraal Mine, 33-15 East VCR stope. This stope has a stope width of 1.3 m and is situated at a depth of about 2900 m below datum. Three panels were part of this monitoring programme, where a strike stabilising pillar was being formed up-dip of and adjacent to the E6 panel and the stope was being mined towards a dyke, against which it stopped. The E5 and E6 panels were mined up-dip due to poor ground conditions but mining was re-established on breast. Good quality backfilling was taking place in the E3 and E4 panels and the same was later introduced to the E5 and E6 panels. The first set (Set 1) of ground motion (PPV) and closure instruments was installed alongside the gully in the E3 panel, followed by a set in the E5 panel, and finally by sets at the top and bottom of the E6 panel. The mining layout and the position of the monitoring sites are shown schematically in Figure 4.3.1.

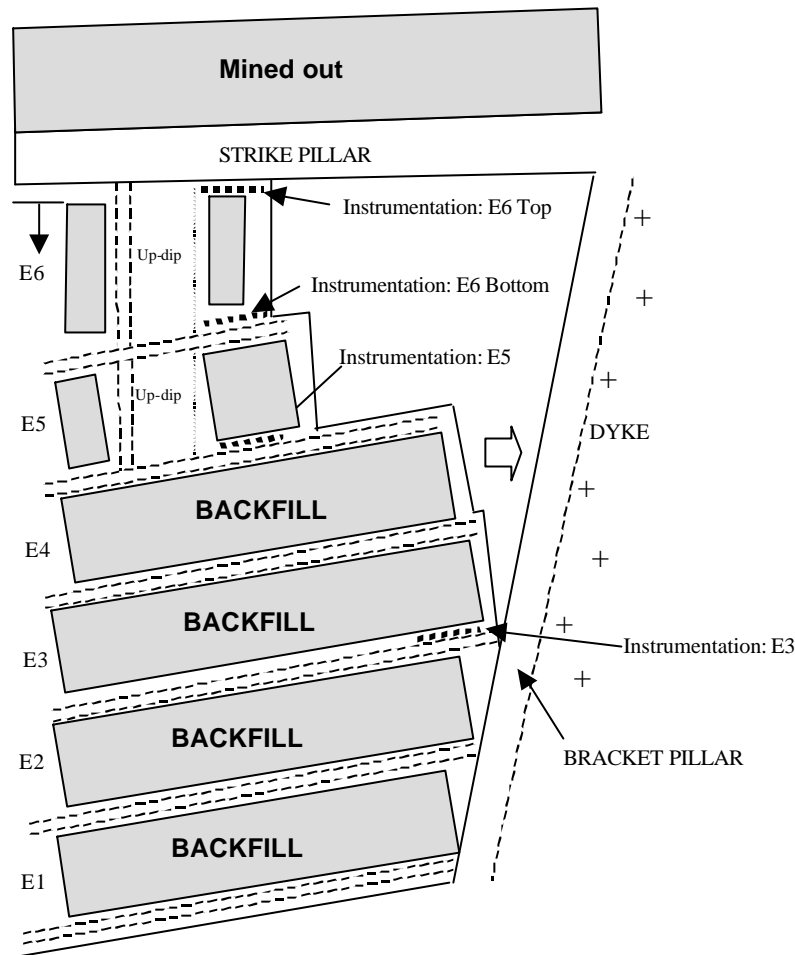


Figure 4.3.1 Schematic of Deelkraal mine monitoring site indicating mining layout and position of installed instrumentation sets

The first panel, panel E3, at the 33-15 East VCR stope was instrumented at the beginning of May 2000. A configuration of quasi-static and dynamic instrumentation was installed in a similar way as that in Tau Tona and Kopanang mines. The E3 panel was mined on breast until it stripped out against the pillar bracketing the dyke, while the E5 and E6 panels were initially re-established by mining up-dip and then on strike.

4.3.1 Review of installed support

Support at this site consists of conventional support components in the working area and along gullies together with backfill. A small amount of bed separation was observed, as was a small fall-of-ground in a gully and panel. The hangingwall condition was considered to be good.

4.3.1.1 Conventional support

Installed conventional support at this site comprised Split Sets in the gullies (2-1-2 pattern), mine poles with headboards as temporary support, pre-stressed elongates (Disc 300 or Eben Haeser MKH 1B) (1.3 m on dip and 1.5 m on strike) together with backfill as permanent support and pre-stressed Durapacks as support on gully sides. Double Durapacks are installed (maximum 2.2 m spacing on strike) on the north side of the gully and single Durapacks on the south side. Backfill is placed up to these packs on the south side and 1 m north of the packs on the north side.

4.3.1.2 Backfill

Backfill of a minimum porosity of 42 per cent is placed between the rows of pre-stressed elongates at a maximum of 4.5 m from the face after the blast. The coefficient of permeability of this backfill is 6.5×10^{-4} , which is a tenth of the permeability of backfill that other mines are achieving. The hyperbolic 'a' and 'b' parameters are 25.926 and 0.324, respectively. The material is well graded with moderately low percentage of fines, with 9.3 per cent at -38 micron and 5.6 per cent at -10 micron. A reduction in the percentage of -38 micron might improve the drainage characteristics of the material. A summary of laboratory tests provides the following:

- It is evident from a laboratory analysis that Deelkraal is producing a well-graded backfill material with perhaps slightly too much -38 micron material.
- The amount of water in the backfill, after a few days, gives water-solids ratios of approximately 20 per cent. This is close to the minimum porosity level for these materials, or the level of porosity found where the stress-strain curve determined is a minimum.
- The RD of the placed backfill appears to improve with time and indicates that it is achieving its objective of providing a stiff support within a few days.

4.3.2 Hangingwall condition

The hangingwall condition at this mine was, where possible, assessed by means of profiling and rock mass rating techniques.

4.3.2.1 Hangingwall profiling

The procedure described in section 3.2 was followed to profile 33-15 East 6 and 5 panels at Deelkraal mine. Measurement of the height of hangingwall steps was carried out on strike at the top, middle and bottom of the panels. Owing to the presence of backfill it was not possible to return to these panels and conduct repeat profiles at the same locations. It was, therefore, not possible to ascertain the deterioration of the hangingwall condition after a period of elapsed time.

At the time of the profiling exercise, the backfill distance from the face at E5 and E6 was 3.6 m and 4.5 m respectively. The immediate face area support comprises mine poles and pre-stressed elongates. Split Sets/grouted rebars are installed in the gully hangingwall. Figure 4.3.2 and Figure 4.3.3 show the profiles from E6 and E5 respectively.

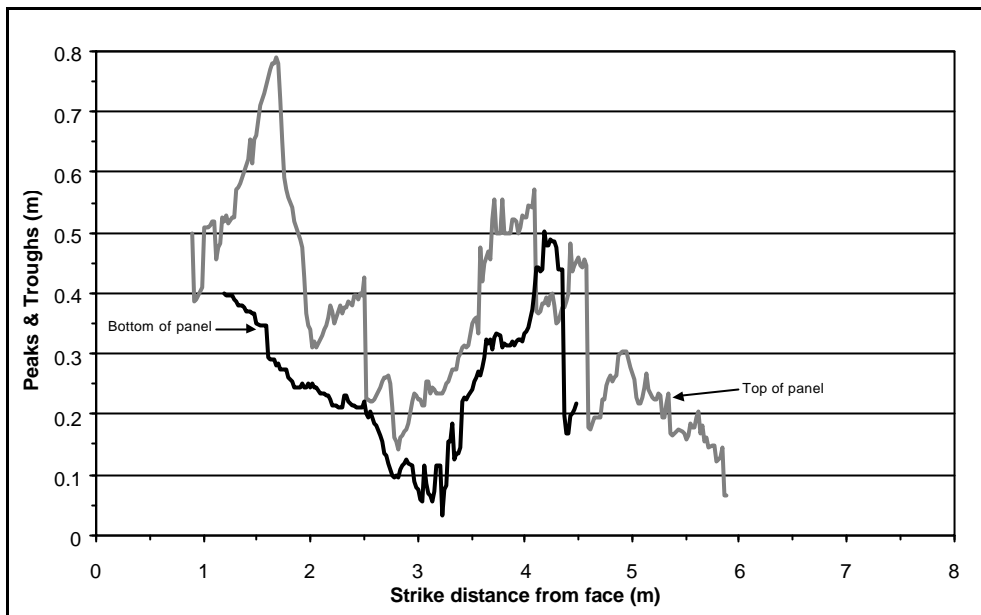


Figure 4.3.2 On-strike hangingwall profile in 33-15 E5 panel at Deelkraal mine

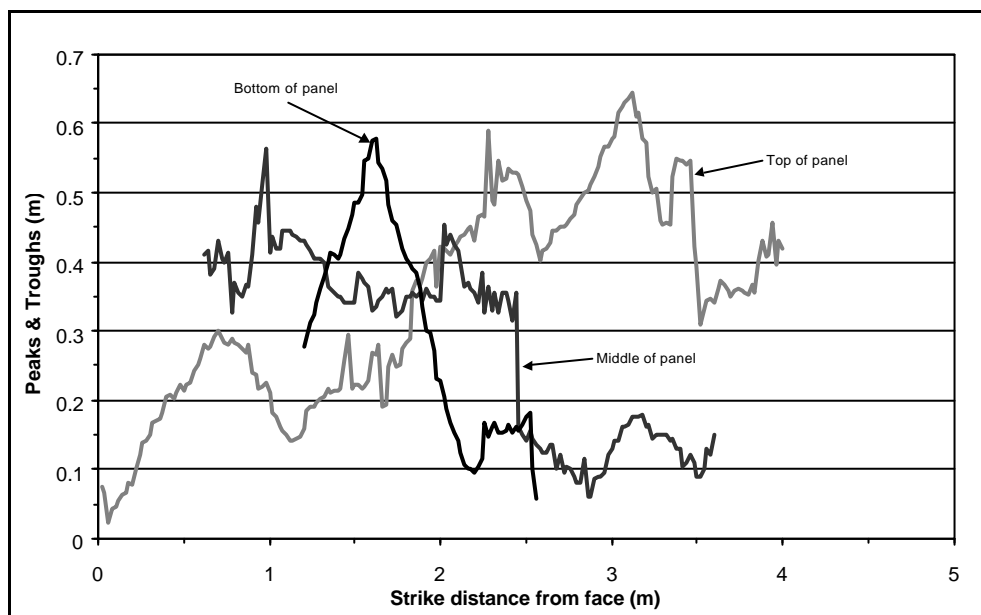


Figure 4.3.3 On-strike hangingwall profile at 33-15 E6 panel at Deelkraal mine

Mapping of hangingwall fractures was also carried out along E6 and E5 panels (see Figure 4.3.4). The same ‘pair’ of fracture sets occurs in both panels but it is rotated to a steeper angle in the lower panel (E5). It was observed that the strike orientation of the fractures is similar for both panels, and is more face-parallel in the middle and lower portions of each panel than in the top portions (cf. Table 4.3.1 and Figure 4.3.5).

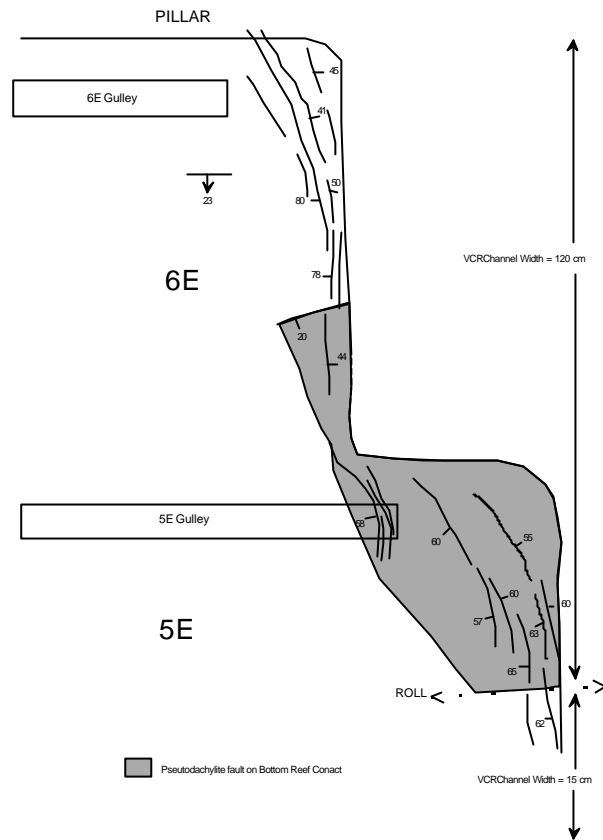


Figure 4.3.4 Sketch of hangingwall fracture orientation in 33-15 E6 and E5 panels at Deelkraal mine

Table 4.3.1 Summary of fracture pattern in 33-15 E6 and E5 panels at Deelkraal mine

Panel	Dip	Strike	Frequency	Persistence	Remarks
6E	45	340	50 / m	4	
6E	80	330	2 / m	3	
6E	41	170	90 / m	4	
6E	20	250	1	1	Pseudotachylite fault from foot
5E	58	175	12 / m	4	
5E	55	320	20 / m	4	
5E	60	340	15 / m	4	
5E	57	175	10 / m	3	
5E	60	148	3 / m	2	
5E	65	178	30 / m	3	
5E	63	175	12 / m	4	
5E	60	170	20 / m	4	

Note: It is assumed that N is at the top of the panel. Therefore, face-parallel fractures strike 0 – 180 and face perpendicular fractures strike 090 – 270.

1 = HIGHLY PERSISTENT and 5 = VERY LOW PERSISTENCE

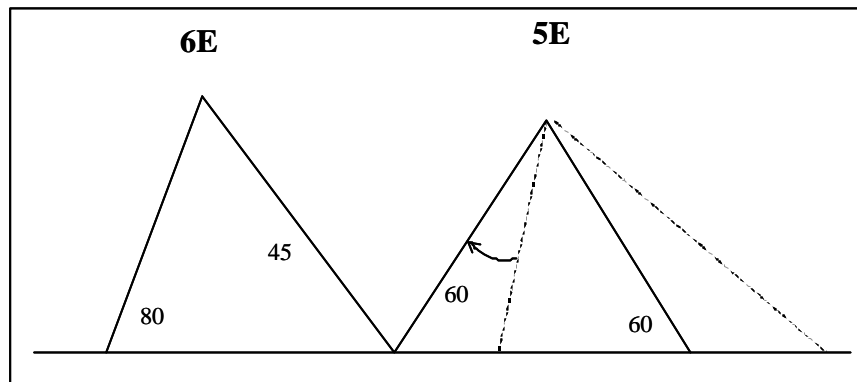


Figure 4.3.5 Comparison of fracture pairs in 33-15 E6 and E5 panels at Deelkraal mine

4.3.3 Hangingwall condition

Rock mass rating measurements (section 3.2.2.1) were carried out to evaluate hangingwall condition at this site. Measurements were, however, difficult because of operational difficulties, and analyses were performed only on data from the gully at the bottom of panel E5 (see Figure 4.3.6). The geotechnical properties of the observed discontinuities are shown in Table 4.3.2.



Figure 4.3.6 Plan showing positions of survey windows in 33-15 stope at Deelkraal mine

Four joint sets were identified and the orientations of these sets are shown in Figure 4.3.7. The strike direction of the fractures shown in Figure 4.3.8 follows the strike direction of joint set 3 (see Figure 4.3.7) and the face (see Figure 4.3.8). However, these fractures have different dip directions, which create the potential to form wedges.

Table 4.3.2 Discontinuity properties measured at Deelkraal mine monitoring site

Discontinuity type	Persistency	Roughness over 0.5 m	Infill type	Infill thickness	Rock type	Spacing
Set 1	4 m	15 mm	Cal	6 mm	Lava	3 m
Set 2	2 m	8 mm	Cal	7 mm	Lava	4 m
Set 3	11 m	8 mm	Cal	4 mm	Lava	1 m
Set 4*	5 m	15 mm	Cal	1 mm	Lava	0.5 m
Fracture 1 & 2	1 m	7 mm	-	-	Lava	

* Set 4 joints were not observed in all the windows.

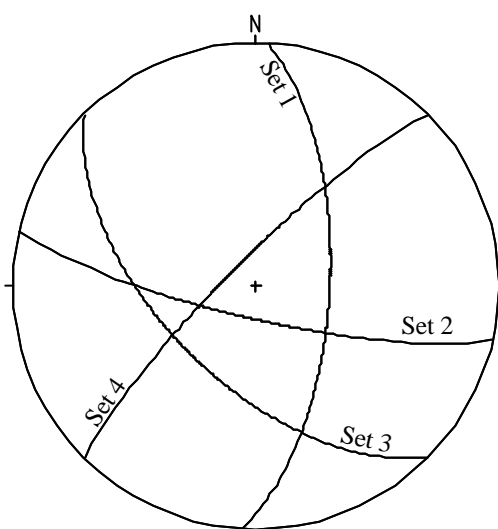


Figure 4.3.7 Equal-area stereo net showing the orientations of the four joint sets in 33-15 E5 panel at Deelkraal mine

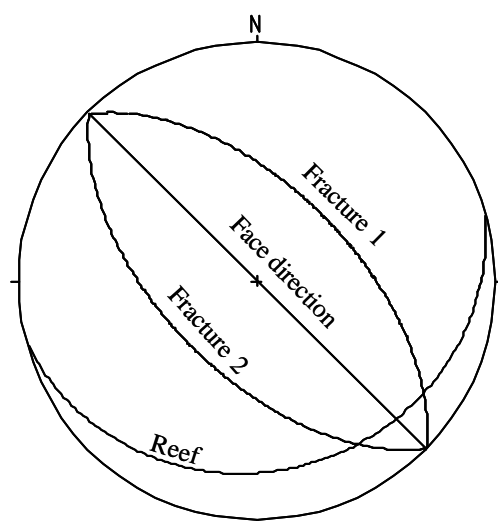


Figure 4.3.8 Equal-area stereo net showing the usual orientations of fracture sets observed in 33-15 E5 panel at Deelkraal mine, compared to the reef dip and face orientation

4.3.3.1 30-31 E4 gully

The results of the survey performed in the gully at the bottom of 33-15 E4 panel could not be analysed in terms of rock mass ratings. No measurements were made in the gully because of the excessive height of the hangingwall but the orientations of the joints and fractures were estimated. The conditions appear to be similar to 'window 1' (15 m) in the E5 gully shown in Table 4.3.3.

4.3.3.2 30-31 E5 gully

The results of the survey performed in the gully at the bottom of 33-15 E5 panel are shown in Table 4.3.3. No trends could be established from the results of the analyses because the orientations of the fractures were different in the two windows. 'Window 2' (53 m) is atypical because the strike direction of the fractures are parallel to the strike of joint set 4. Joint set 4 is also more prominent in this window.

Table 4.3.3 Results of geotechnical survey in 33-15 E5 gully at Deelkraal mine

Distance from face	Window length	RQD	RMR'	Q'	Average spacing of fractures	No. of Fracture 1	No. of Fracture 2
15 m	10 m	11.4%	63	1.80	102 mm	75	23
53 m*	5 m	10.0%	63	1.58	136 mm	10	1

*The strike orientation of the fractures in this window is parallel to joint set 4 but the dip angles are similar to those of the other fractures.

4.3.4 Ground motion and closure

The seismic data recorded in panel E3 is analysed in respect of peak particle velocities. The PPVs recorded close to the face, in the earliest stage of the backfill, are compared to the PPVs recorded further back where the backfill is expected to provide better support. The results are presented in Figure 4.3.9.

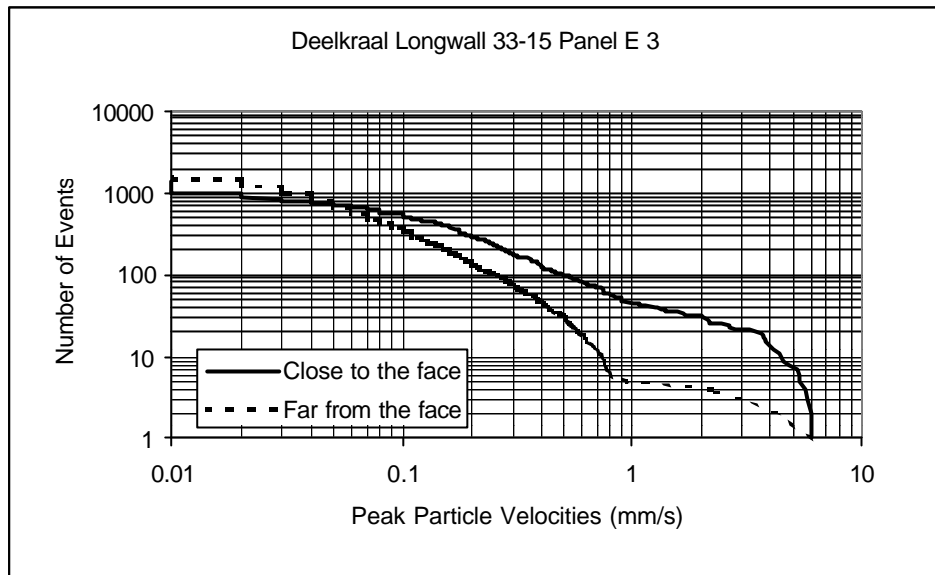


Figure 4.3.9 Peak particle velocities recorded close to the face (upper line) and far from the face (lower line) in 33-15 E3 panel at Deelkraal mine

It is evident from Figure 4.3.9 that the PPVs recorded close to the face are higher than the PPVs recorded further from the face in the area where the backfill is more consolidated and provides higher support resistance.

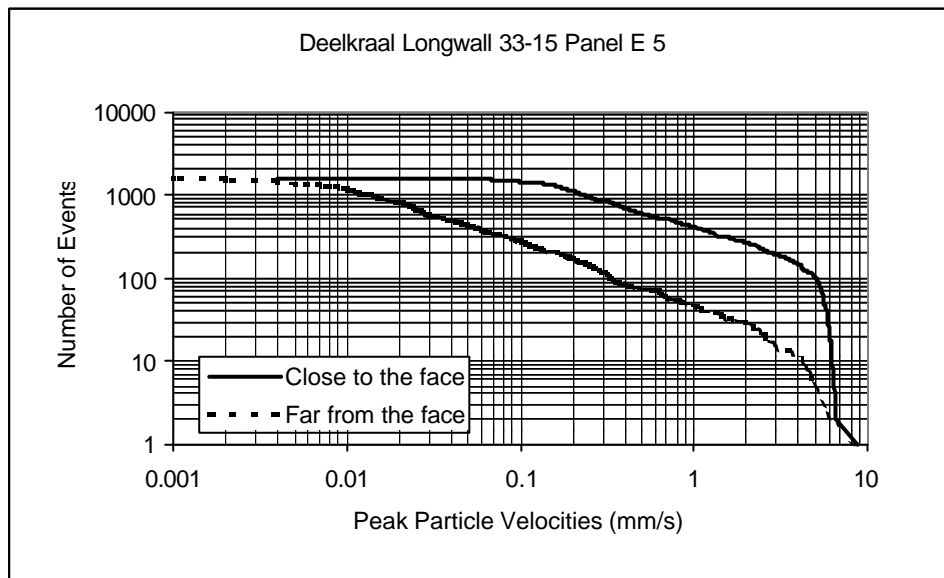


Figure 4.3.10 Peak particle velocities recorded close to the face (upper line) and far from the face (lower line) in 33-15 E5 panel at Deelkraal mine

It can be seen in Figure 4.3.10 that the PPVs recorded close to the face (where backfill has recently been placed and is at its earliest stage of loading) are higher than the PPVs recorded further back in the area where the backfill provides better support. This is a similar result to that obtained for panel E3 located in the same longwall, the only difference being that the PPVs recorded in panel E5 are generally higher than the PPVs recorded in panel E3.

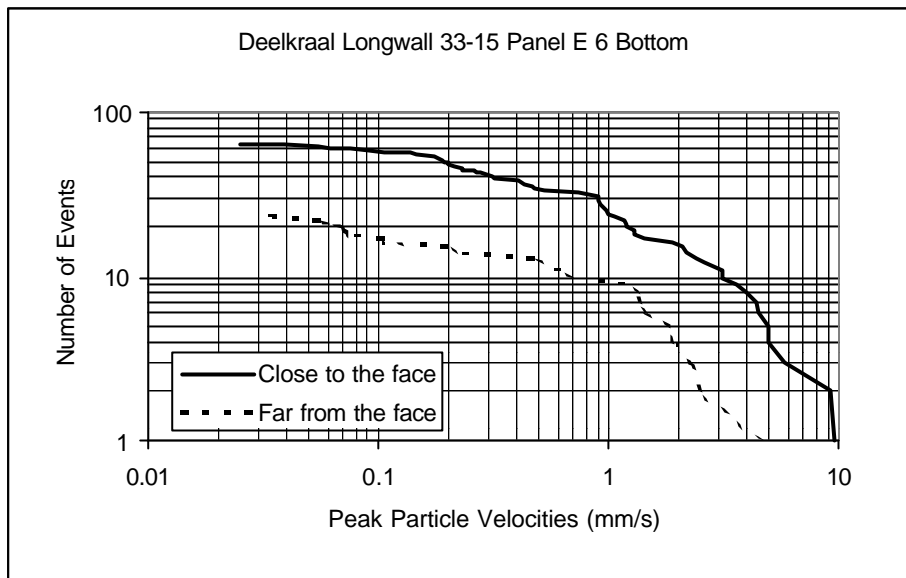


Figure 4.3.11 Peak particle velocities recorded close to the face (upper line) and far from the face (lower line) in 33-15 E6 Bottom panel at Deelkraal mine

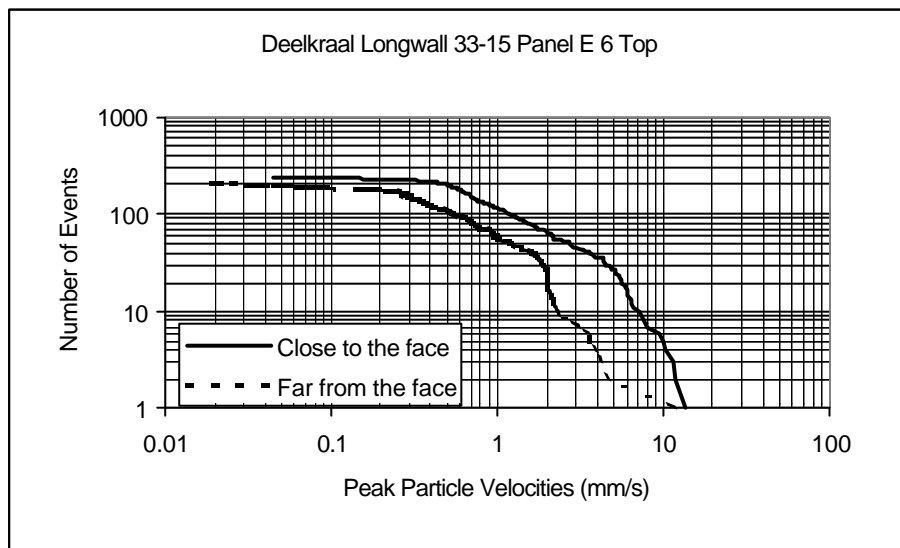


Figure 4.3.12 Peak particle velocities recorded close to the face (upper line) and far from the face (lower line) in 33-15 E6 Top panel at Deelkraal mine

It is clearly indicated in Figure 4.3.9, Figure 4.3.10, Figure 4.3.11 and Figure 4.3.12 that the PPVs recorded close to the face adjacent to the recently placed backfill are higher in all monitored panels than the PPVs recorded further back in the area where the backfill is already consolidated and provides better support. The PPVs recorded in the different panels were, however, influenced by the site conditions and exhibit different patterns.

The PPVs recorded in the top and bottom of the panel E6 are shown in Figure 4.3.13. The top of the E6 panel is adjacent to the strike pillar. A seismic channel, especially designed to improve the recording capabilities of the system was used in this study.

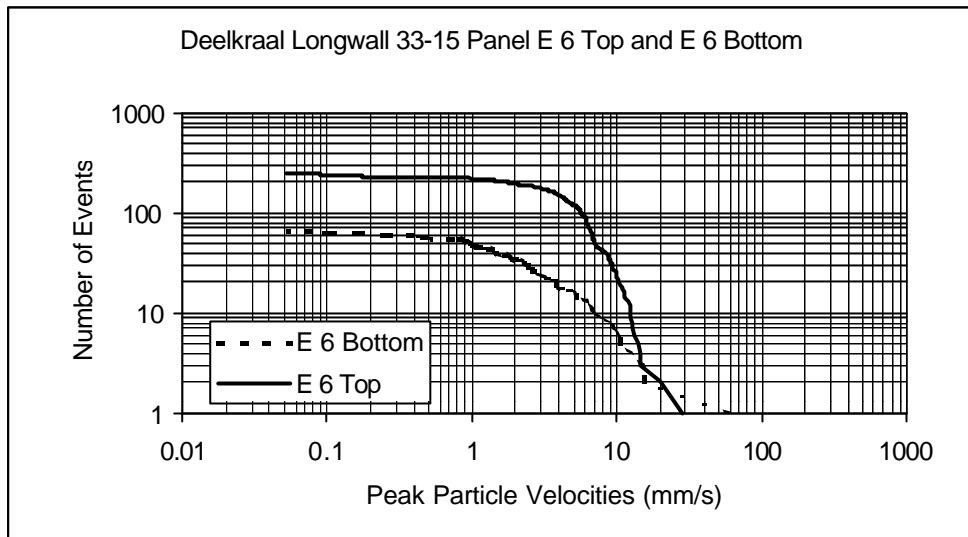


Figure 4.3.13 Peak particle velocities recorded at the top and bottom of 33-15 E6 panel at Deelkraal mine

Figure 4.3.13 clearly illustrates that close to the pillar (i.e. E6 Top) the number of smaller events (up to 10–11 mm/s) is greater than the number of smaller events further away from the pillar (i.e. E6 Bottom). Conversely, there are more high velocity events (12-60 mm/s) further from the pillar. This effect was also seen at Tau Tona mine.

The PPVs recorded in panels E3 and E5 are shown in Figure 4.3.14. In this case, panel E3 is adjacent to old backfilled panels, e.g. panel E2, which were stopped against the dyke lying ahead of the stope (Figure 4.3.1). Panel E5 is located in the middle of the long wall and is surrounded by actively mined areas but also by well compacted backfill in panel E4.

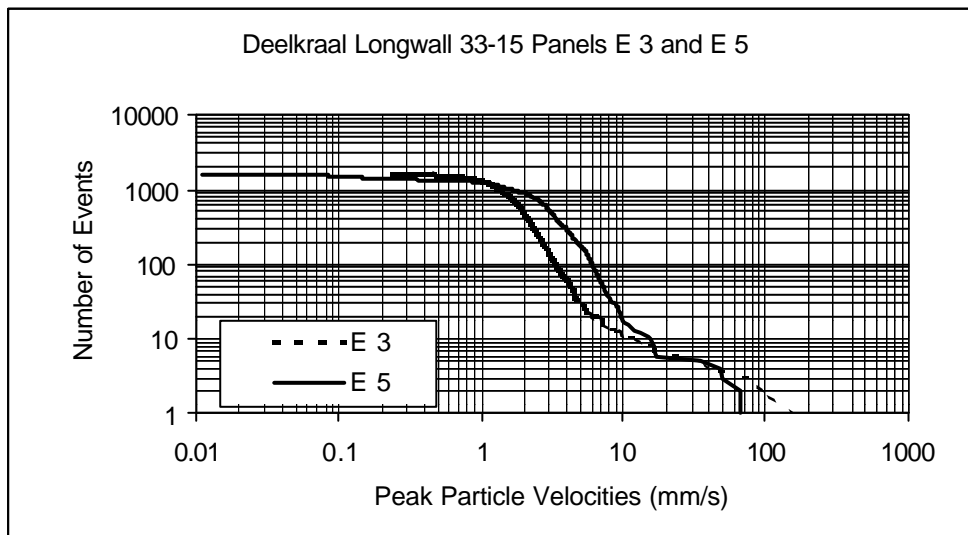


Figure 4.3.14 Peak particle velocities recorded in E5 and E3 panels at Deelkraal mine

It is interesting to note that contrary to the differences in the PPVs between E6 Top and E6 Bottom, the PPVs recorded in E3 panel are lower than the PPVs recorded in E5 panel. As the closure is a continuous process, even in the back area, the old backfill in the panels adjacent to the E3 has been consolidated to such a degree that its effect on the panel is similar to that expected from an abutment.

Closure measurements were also recorded by continuous closure meters in each panel. The results are listed in Table 4.3.4.

Table 4.3.4 *Rate of closure measured in E3, E5, E6 Top and E6 Bottom panels at Deelkraal mine*

Panel Number	Closure rate (mm/day)
E3	6.1
E5	9.8
E6 Top	4.0
E6 Bottom	5.5

It is evident from Table 4.3.4 that in both pairs of comparable panels (E3 and E5, and E6 Top and E6 Bottom) the closure rate measured close to the regional support, i.e. panels E3 and E6 Top, is less than the closure measured in the middle of the longwall, i.e. panels E5 and E6 Bottom. This result is confirmed by an analysis of the dynamic closure as a function of PPVs.

The support behaviour (i.e. dynamic closure) in each panel is plotted as a function of PPVs in Figure 4.3.15, Figure 4.3.16, Figure 4.3.17 and Figure 4.3.18. Negative values of dynamic closure indicate a state of opening during the seismic loading of the stope.

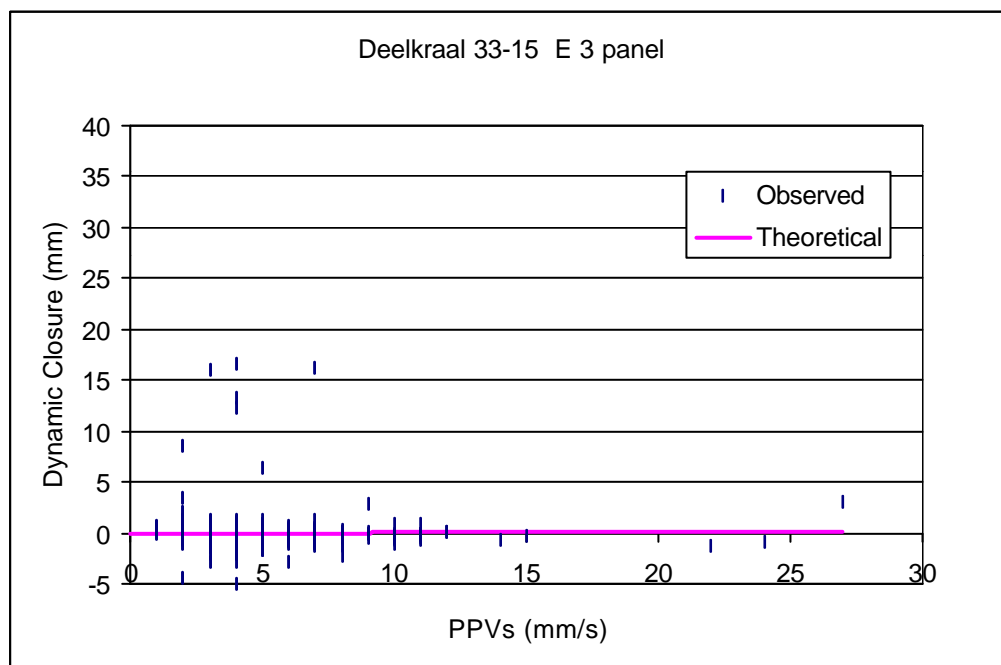


Figure 4.3.15 *Dynamic closure as a function of PPVs recorded in E3 panel at Deelkraal mine*

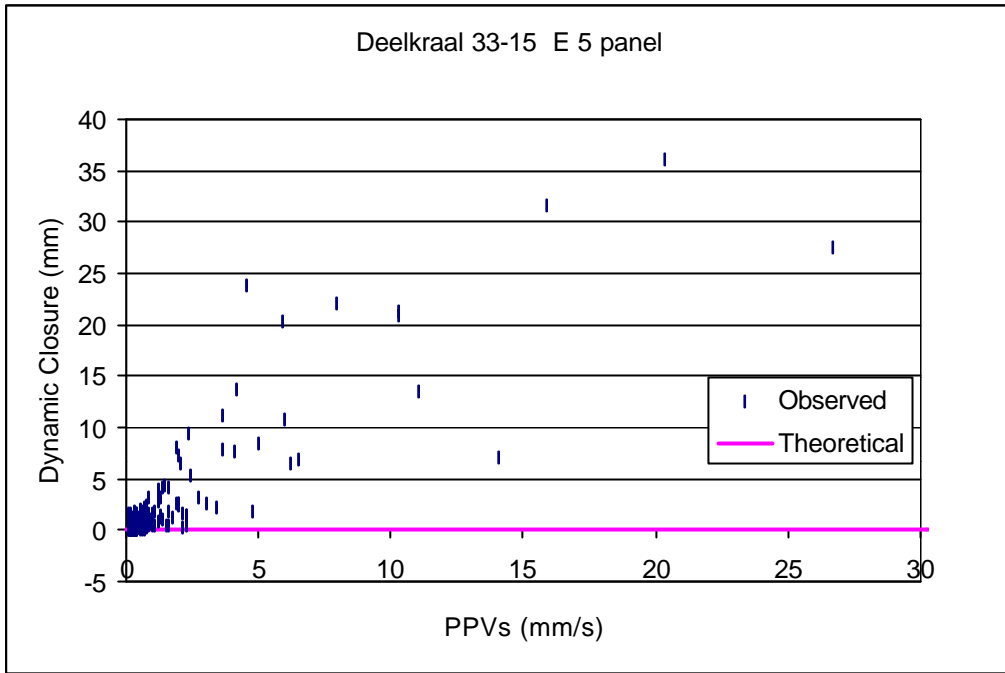


Figure 4.3.16 Dynamic closure as a function of PPVs recorded in E5 panel at Deelkraal mine

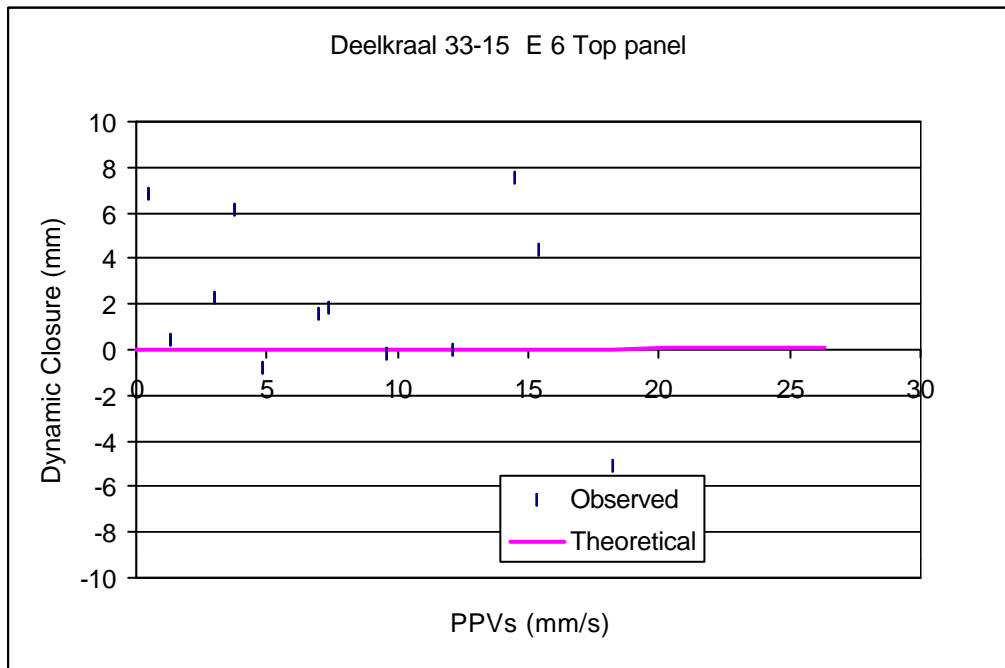


Figure 4.3.17 Dynamic closure as a function of PPVs recorded in E6 Top panel at Deelkraal mine

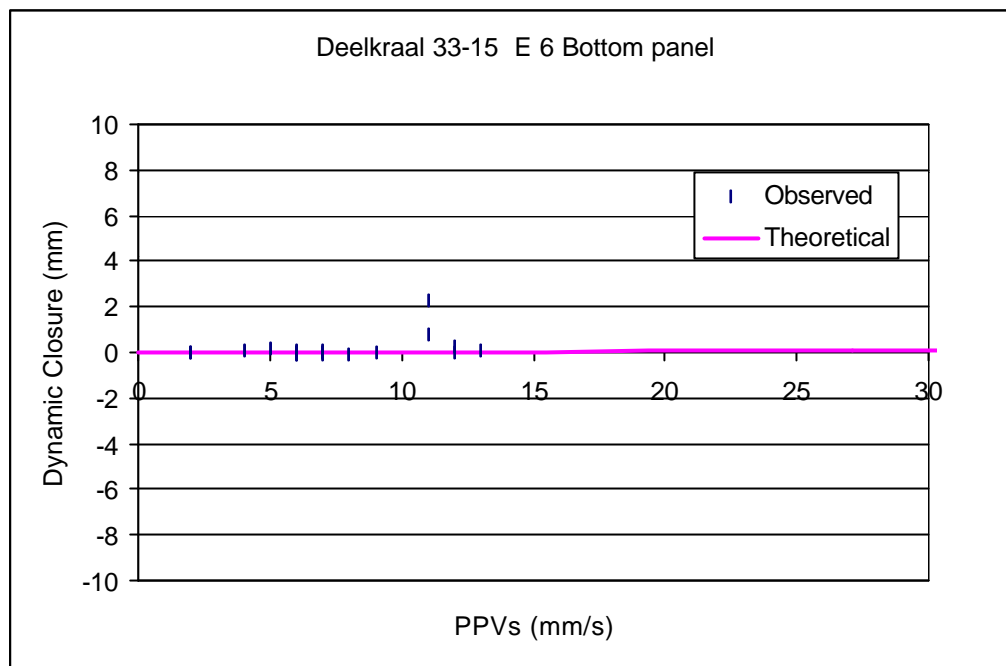


Figure 4.3.18 *Dynamic closure as a function of PPVs recorded in E6 Bottom panel at Deelkraal mine*

It is evident, that the dynamic closure in areas close to the abutment (in this case old, well consolidated backfill) in panel E3 (cf. Figure 4.3.15) and close to the pillar in panel E6 Top (cf. Figure 4.3.17) is less influenced by the PPVs. Areas away from the abutments, i.e. panels E5 (cf. Figure 4.3.16) and E6 Bottom (cf. Figure 4.3.18), in the middle of the stope, show an increase in dynamic closure with an increase in PPVs. Therefore, areas adjacent to regional support could potentially require local support units of a lower energy absorption characteristic than areas further away from the regional support and nearer the middle of the stope. However, the increase in microseismicity associated with the increase in the average pillar stress (which results from face advance) should also be taken in to account.

The dynamic closure in all analyses shown above was taken at a point close to the face where backfill was not fully consolidated and the support it provides can be considered to be local rather than regional. The dynamic closure obtained far from the face in the areas of well-consolidated backfill shows a similar behaviour to panels E3 and E6 Top (Figure 4.3.15 and Figure 4.3.17) where a strong influence of the regional support exists. This fact indicates that far from the face the support provided by the backfill is similar to that of the regional pillar support.

4.3.5 Numerical modelling

Figure 4.3.19 and Figure 4.3.20 show the vertical stress modelled along a transect line in the strike stabilising pillar adjacent to the monitoring site at Deelkraal mine as a function of monthly face advance, for backfilled and non-backfilled scenarios respectively. The effect of the backfill is reflected as a 9 per cent reduction in pillar stress. For comparative purposes, Figure 4.3.21 summarises the initial and final vertical stress profiles along the stabilising pillar for backfill and non-backfill cases. Figure 4.3.22 shows the modelled build-up of vertical stress with increasing face advance at a fixed position in the pillar adjacent to a corresponding monitoring position, for backfilled and non-backfilled cases.

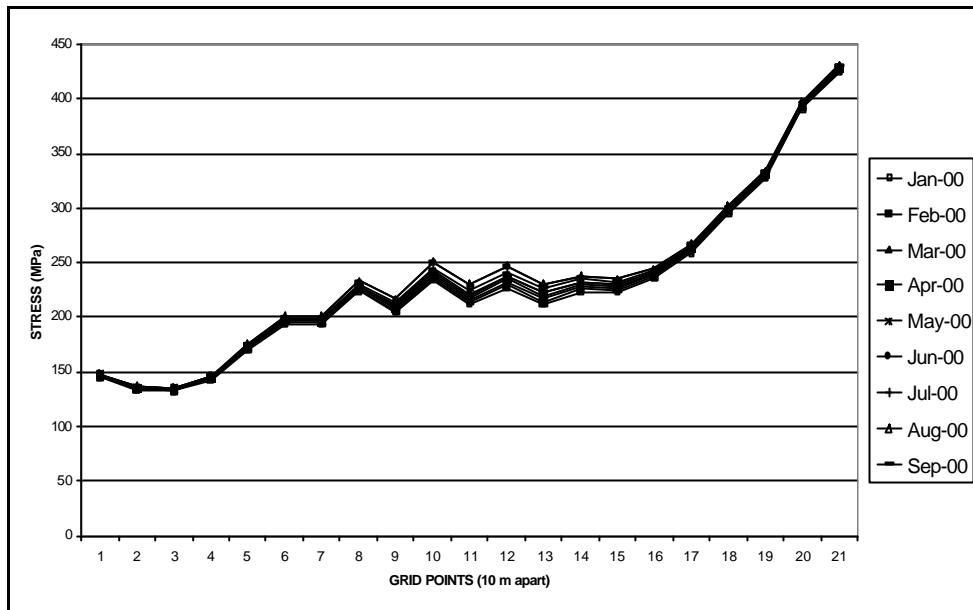


Figure 4.3.19 *Modelled vertical stress profiles along strike stabilising pillar with backfill for given monthly face position at Deelkraal mine*

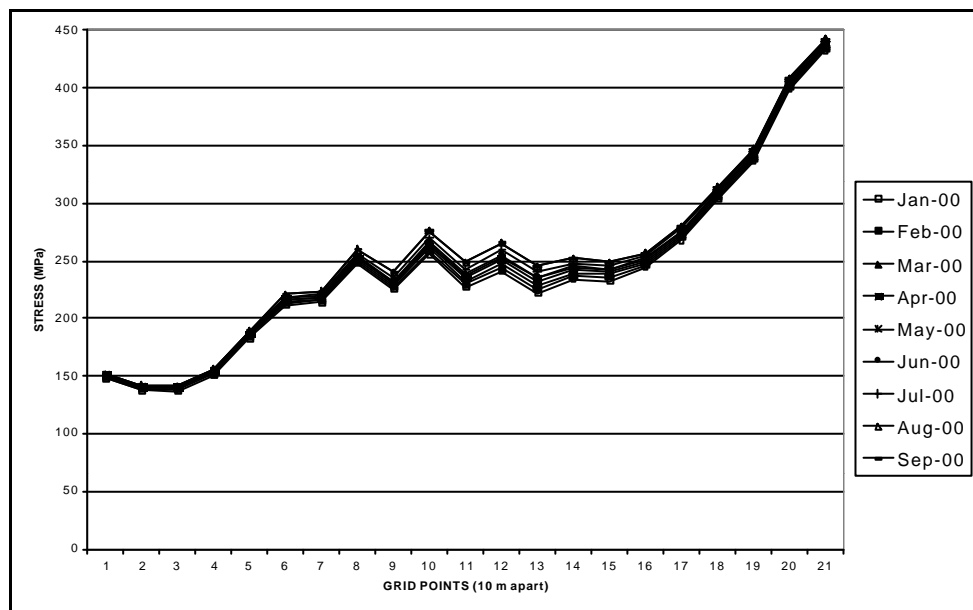


Figure 4.3.20 *Modelled vertical stress profiles along strike stabilising pillar without backfill for given monthly face position at Deelkraal mine*

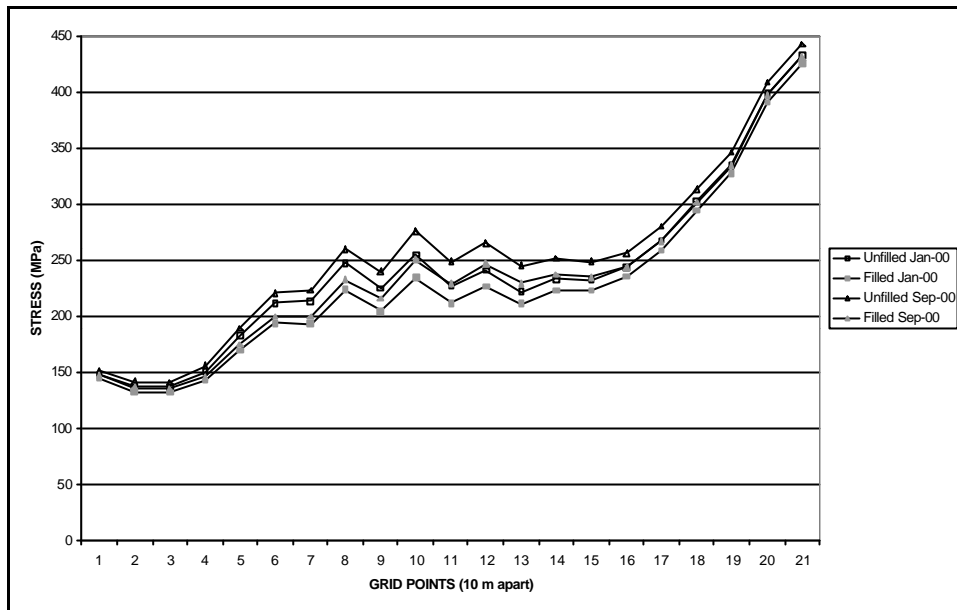


Figure 4.3.21 Comparison of initial and final modelled vertical pillar stress profiles with and without backfill at Deelkraal mine

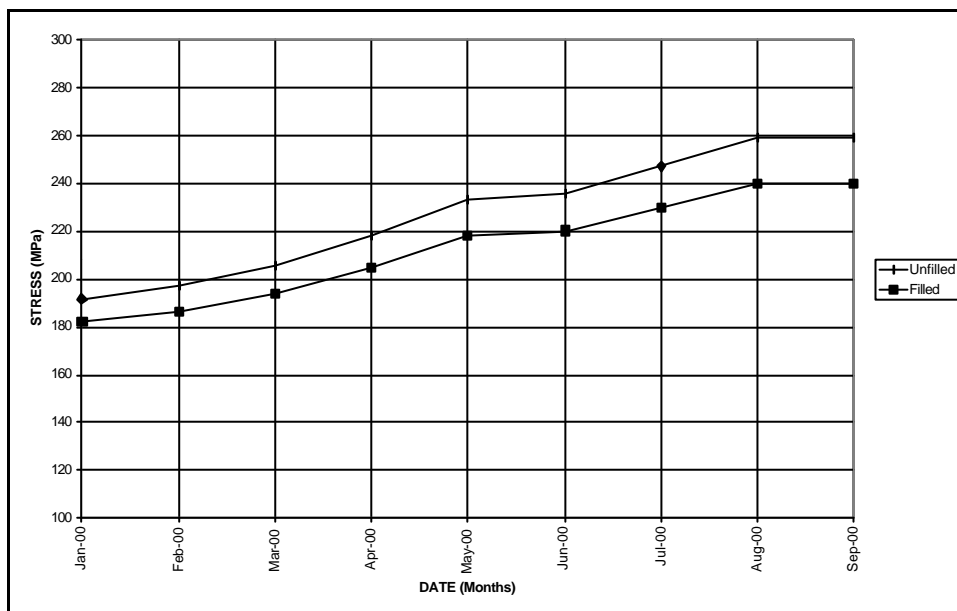


Figure 4.3.22 Modelled vertical pillar stress at fixed location as a function of monthly face advance with and without backfill at Deelkraal mine

4.3.6 Summary

The key findings from the Deelkraal site can be summarised as:

- Face parallel fractures close to the strike pillar are, in general, dipping at steeper angles than equivalent fractures further away from the strike pillar.
- The deterioration of hangingwall condition over time could not be determined due to the presence of backfill.
- PPVs recorded close to the face adjacent to the recently placed backfill are higher in all monitored panels than PPVs recorded further back in the areas where the backfill

is already consolidated and provides greater support. Similar behaviour was found for the backfill sites at Kopanang and Tau Tona. Differences from site to site are the result of variations in the local conditions.

- The dynamic closure in areas close to the abutment (in this case old, well consolidated backfill) and in areas close to the pillar is less influenced by PPVs. Areas away from the abutments and pillars, in the middle of the stope, show an increase in dynamic closure with an increase in PPVs.
- Numerical modelling indicates that backfill reduces strike pillar stress by almost 9 per cent at this site.

4.4 Mponeng mine

A site for the monitoring of a ‘VCR dip pillar without backfill’ mining layout was established at Mponeng mine, Western Deep Levels, in the 94-48/99-48 stope at a depth of between 2550 m and 2700 m. Stope widths of between 1.1 m and 1.2 m were recorded. This stope was being mined eastwards towards a cut-off position that would create a dip pillar against the existing mined out area of the 94-49/99-49 stope (Figure 4.4.1). A set of instruments (Set 1) was installed in the 94-48, E1 panel. It was intended that, when up-dip mining and re-establishment of the 99-48, E8, E9 and E10 panels was complete, additional sets of instruments would be installed. However, before this could take place a rockburst occurred that caused severe damage to these panels and prevented further access to the stope. No backfill was placed in these panels.

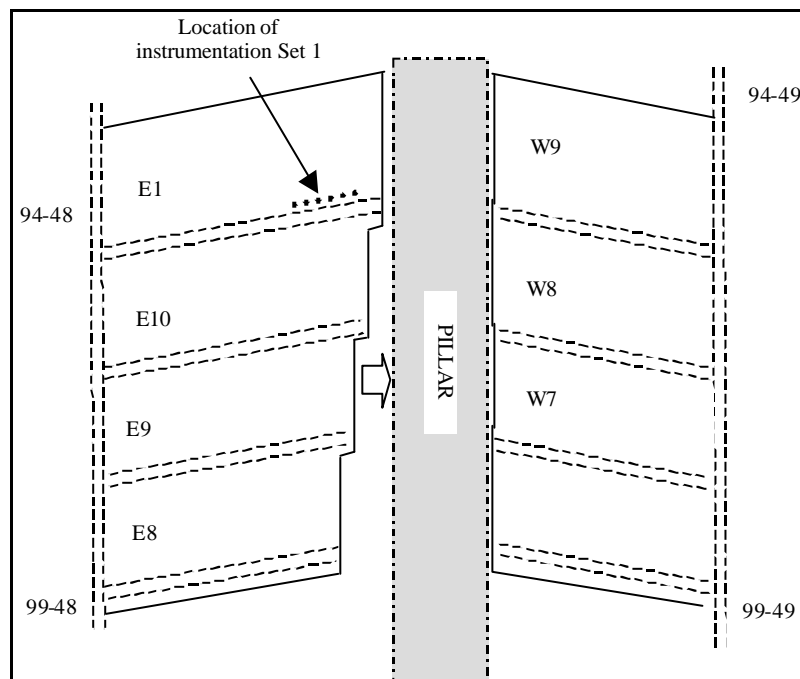


Figure 4.4.1 Schematic of Mponeng 94-48 E 1 ‘dip pillar’ monitoring site

4.4.1 Review of installed support

Support at this site consists only of conventional support components, no backfill was placed in this stope. A small amount of bed separation was noticed in this panel. However, hangingwall and footwall condition was observed to be good.

4.4.1.1 Conventional support

Temporary face support consisted of 150 mm diameter mine poles spaced at 1.4 m on strike and 1.2 m on dip. Permanent stope support consisted of 0.75 m x 0.75 m composite packs at a skin-to-skin spacing of 1.6 m or mechanical props with headboards (see Figure 4.4.3). Double packs (1.1 m x 0.75 m) were installed on both gully sides and the gully hangingwall supported either with 1.2 m rockbolts on 2x1 pattern at 1 m spacing or two lines of Split Sets.

4.4.2 Hangingwall condition

The hangingwall at the bottom of the E1 panel exhibited dominant backward-dipping (B) fracturing of 60°-65° with coarse stope-normal hackle lineation (Figure 4.4.2). Further up dip in the panel the dominant fracturing was forward-dipping (F) at 60°-70°, with some 65° backward-dipping fracturing (B) (Figure 4.4.3). Mid-panel the fracturing consisted of a mixture of closely spaced 70° forward-dipping, vertical and 80° backward-dipping fractures (Figure 4.4.4). Towards the top of the panel, steeper (70°-80°) and more widely spaced forward-dipping fractures existed, with some long low-inclination fractures noted between these (Figure 4.4.5).



Figure 4.4.2 *Hangingwall at bottom of E1 panel, 94-48 VCR stope at Mponeng mine: exhibiting dominant backward-dipping fracturing (B) (face at right)*



Figure 4.4.3 Hangingwall up dip from the bottom of E1 panel, 94-48 VCR stope at Mponeng mine: exhibiting dominant forward-dipping fracturing (F) and minor backward-dipping fracturing (B) (face at left)



Figure 4.4.4 Hangingwall at middle of E1 panel, 94-48 VCR stope at Mponeng mine (face at left)



Figure 4.4.5 Hangingwall at top of E1 panel, 94-48 VCR stope at Mponeng mine (face at right)

4.4.3 Ground motion and closure

In Figure 4.4.6 the PPVs recorded close to the face in the E1 panel are compared to those recorded far from the face. It is evident from Figure 4.4.6 that the peak particle velocities recorded close to the face are higher than the peak particle velocities recorded far from the face.

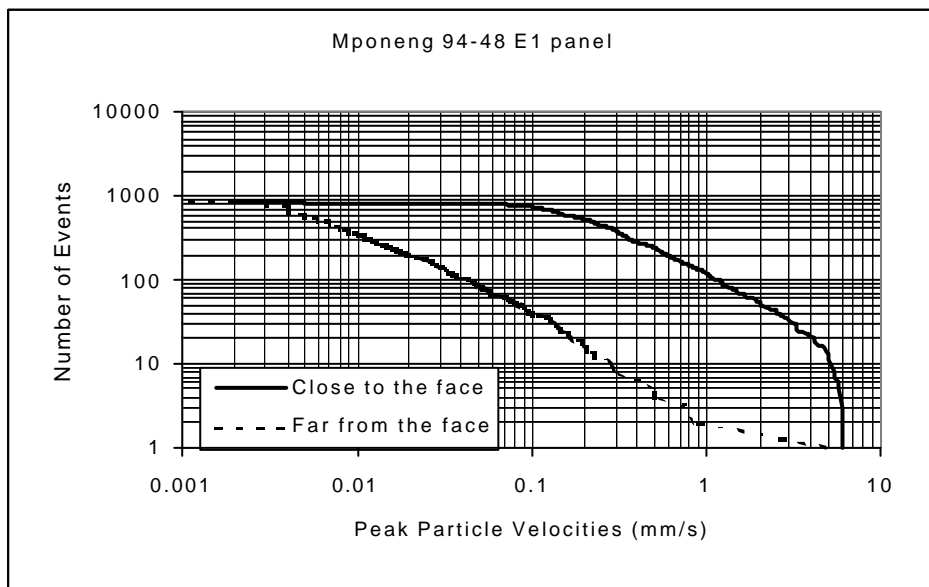


Figure 4.4.6 Peak particle velocities recorded close to the face (upper line) and far from the face (lower line) in panel 94-48 E1 at Mponeng mine

Both the theoretical and observed dynamic closure are plotted against PPVs for this site (Figure 4.4.7).

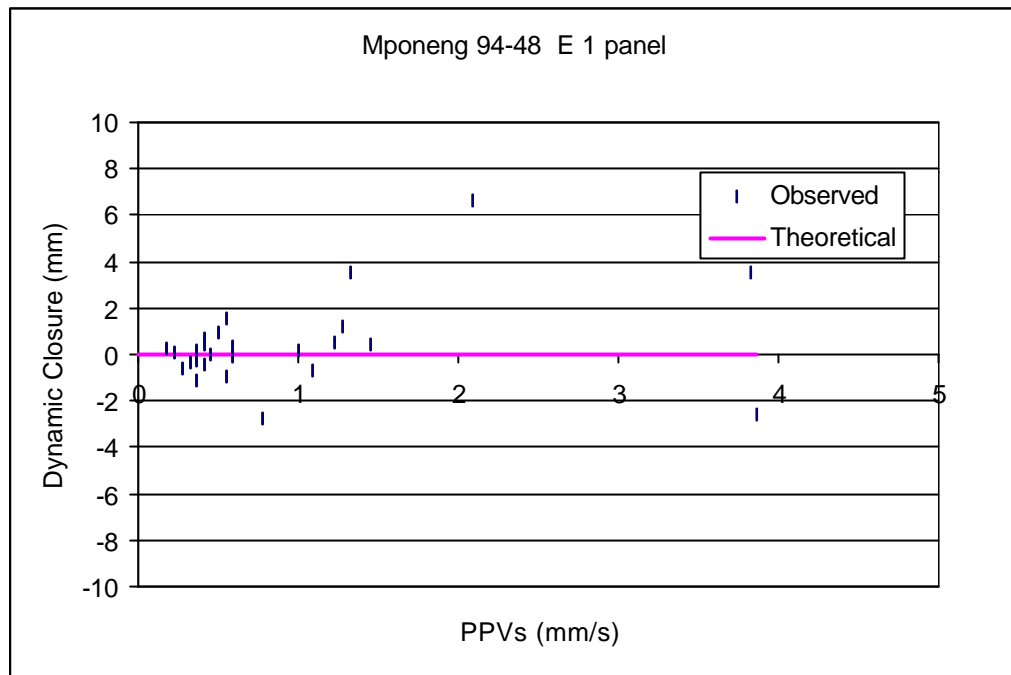


Figure 4.4.7 *Dynamic closure as a function of PPVs obtained for E1 panel at Mponeng mine*

An increase in dynamic closure with an increase in PPVs is noticeable in Figure 4.4.7. This type of behaviour is also observed at Tau Tona and Deelkraal in the panels located in the middle of the longwall and that are, therefore, less influenced by the regional support. This region is, therefore, categorised as having relatively low resistance to dynamic closure. In this particular case, however, it is not clear whether the dip stabilising pillar scenario or the absence of backfill plays a dominant role. The results from Driefontein mine, which has a similar mining layout but also includes backfill, are discussed in a subsequent section (section 4.5.3).

4.4.4 Numerical modelling

Figure 4.4.8 shows (for the prevailing non-backfilled scenario) the vertical stress modelled, as a function of monthly face advance, along a transect line that crosses the final extent of dip stabilising pillar ahead of the monitored E1 panel at Mponeng mine. Figure 4.4.9 shows the modelled build-up of vertical stress with increasing face advance at a fixed point in the pillar ahead of the position that corresponds to the ground motion/closure monitoring station in the E1 panel.

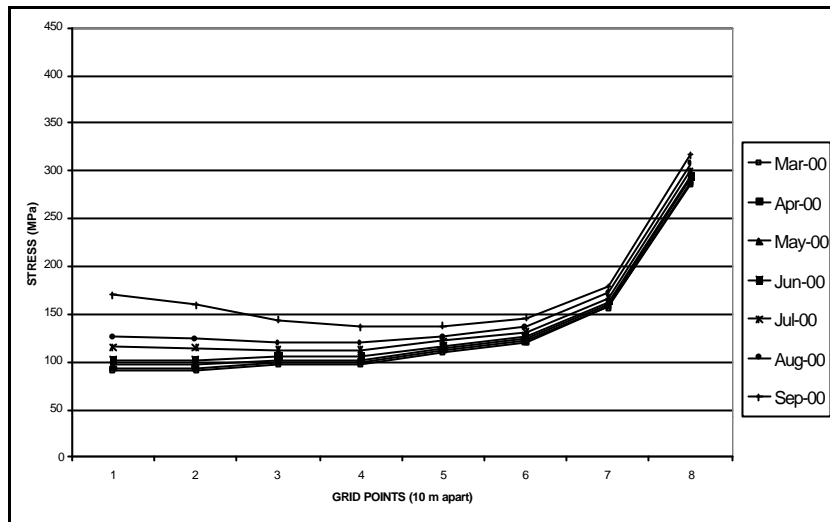


Figure 4.4.8 *Modelled vertical stress profiles across dip stabilising pillar without backfill for given monthly face position at Mponeng mine*

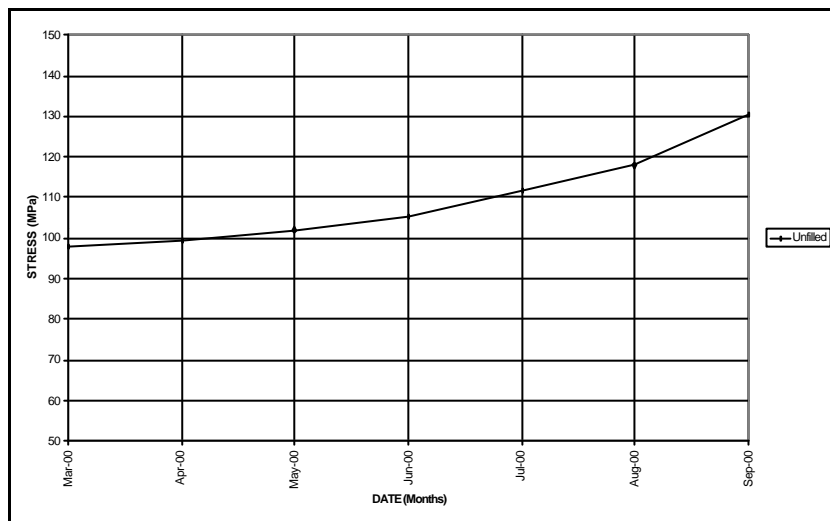


Figure 4.4.9 *Modelled vertical pillar stress at fixed location in dip pillar as a function of monthly face advance without backfill at Mponeng mine*

4.4.5 Summary

The key findings from the Mponeng site can be summarised as:

- No significant interpretation could be made from the fracturing observed at this site.
- Peak particle velocities recorded close to the face are higher than the peak particle velocities recorded far from the face.
- There is an increase in dynamic closure with an increase in PPVs. This type of behaviour is also observed at Tau Tona and Deelkraal in the panels located in the middle of the longwall and that are, therefore, less influenced by the regional support. In this particular case, however, it is not clear whether the dip stabilising pillar scenario or the absence of backfill plays a dominant role.
- Numerical modelling indicates that pillar stress increased by approximately 30 MPa at a given point ahead of the advancing face over the monitoring period.

4.5 Driefontein 5 Shaft

The 'dip stabilising pillar with backfill' scenario was studied in the 5 Shaft area of Driefontein Gold Mine, where closely spaced dip pillars and backfill provide regional support. The 50-25 stope, panel E5 (Figure 4.5.1) on the Carbon Leader Reef at an approximate depth of 3400 m, was chosen as the monitoring site.

The closely spaced dip pillar with backfill scenario is designed for 140 m mining spans with 40 m wide dip pillars.

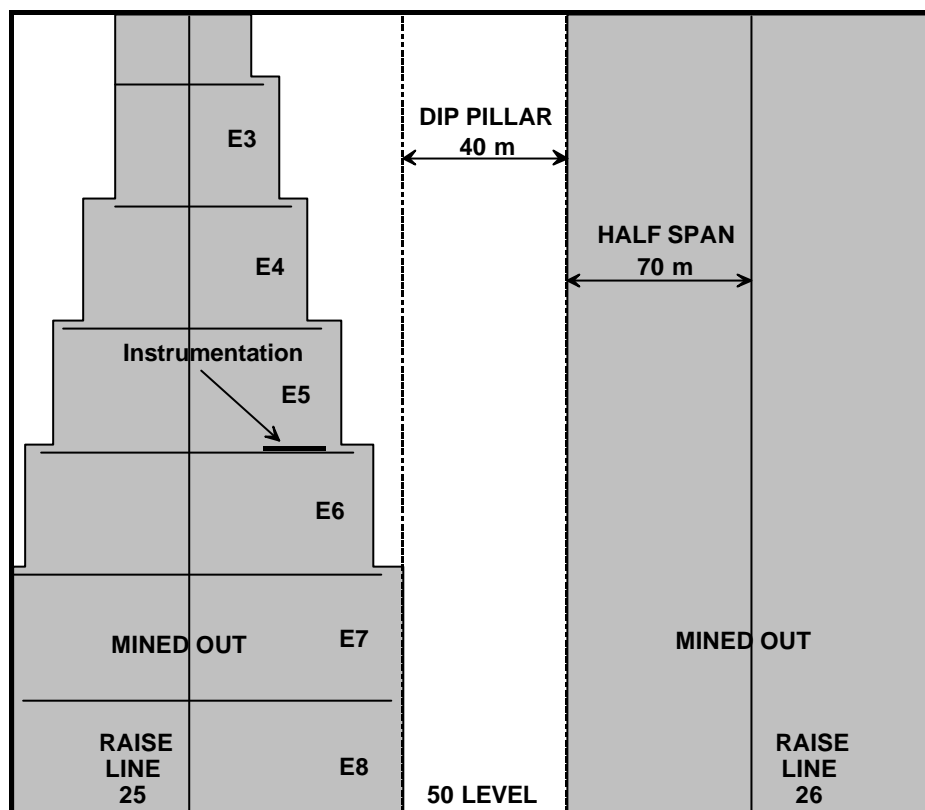


Figure 4.5.1 Schematic of monitoring site and instrumentation layout at Driefontein 5 Shaft

4.5.1 Review of installed support

Support at this site consists of conventional support components and backfill. Good hangingwall conditions were observed in the face area.

4.5.1.1 Conventional support

Stope support consists of rows of pre-stressed pipe sticks (between which backfill is placed) and mechanical props with headboards as temporary face support. On the up-dip gully edge 1.2 m x 1.2 m timber packs are installed and 2 m up-dip of these another line of packs is installed. A line of 4x4 (ft) packs is also installed at the top of the panel on the down-dip edge of the upper gully. Packs are pre-stressed using packsetters.

4.5.1.2 Backfill

Backfill of a minimum porosity of 43.7 per cent is placed between the rows of pre-stressed pipe sticks, with a gap of 1 m being maintained from the lower edge of the backfill to the second line of packs to prevent packs from (allegedly) being pushed out by the backfill. The coefficient of permeability of this backfill is 5.2×10^{-4} , which is a tenth of the backfill permeability achieved by many mines. The hyperbolic 'a' and 'b' parameters are 19.5 and 0.412, respectively. The material is well graded with an above average percentage of fines, with 18 per cent at -38 micron and 7 per cent at -10 micron. A reduction in the percentage of -38 micron would improve the drainage characteristics of the material. A summary of laboratory tests provides the following:

- The above average percentage of -10-micron material in the backfill (average 7 per cent) is producing a backfill that drains poorly.
- The amount of water in the backfill after 2 days was a high 17 per cent at the bottom and middle of the bag, while the top of the bag produced an acceptable 10 per cent.
- The RD of the placed backfill, measured using the constant volume technique, indicates that the density of placed backfill after 2 days was on average 1.8.

The backfill appeared to be making good contact with the hangingwall. However, the distance being maintained away from gully packs means that in a 25 m panel, only 19 m of panel length is backfilled. Backfill drainage was also identified as a problem at this site, resulting from a combination of high fines content and heavy geotextile material.

4.5.2 Hangingwall condition

Hangingwall conditions in the face area were observed to be good and this was attributed to a steep dipping joint pattern, running parallel with the face.

The assessment of gully hangingwall conditions by means of rock mass ratings, was carried out at discrete positions along several gullies in the 50-25 stope, including the instrumented (i.e. ground motion and closure monitoring) E5 gully. The aim of the exercise was to determine a time dependent deterioration of the hangingwall by performing the surveys at various intervals down a gully, i.e. in rock that had been exposed for different periods of time. It was, however, not possible to get into gullies below panel E7 (see Figure 4.5.2) and, therefore, the analysis was restricted to panels E4, E5, E6 and E7.

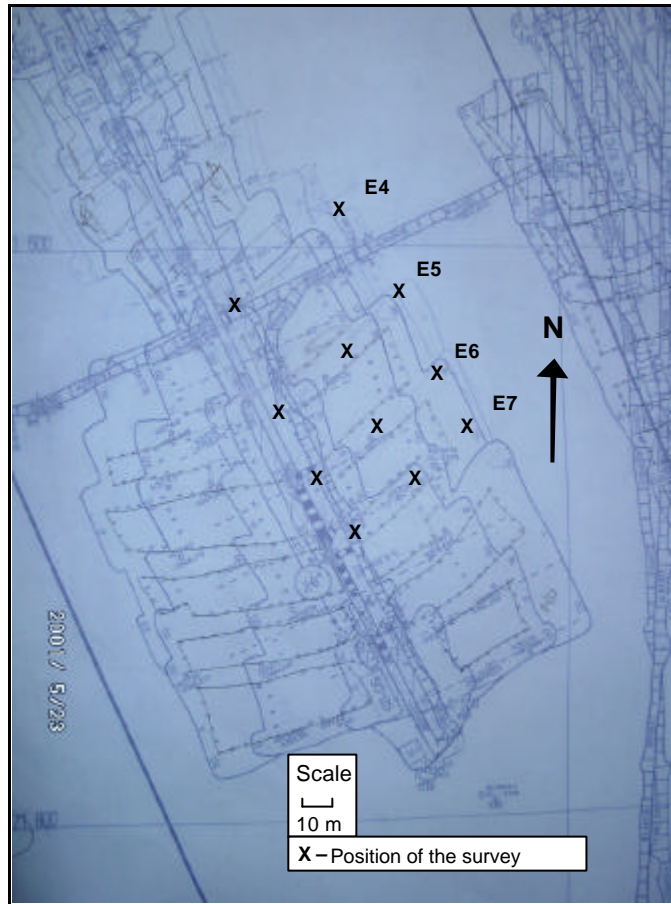


Figure 4.5.2 Plan of Driefontein 5 Shaft site showing positions of survey windows

In general, hangingwall conditions in the 50-25 stope at Driefontein 5 Shaft were relatively good as shown in Figure 4.5.3.

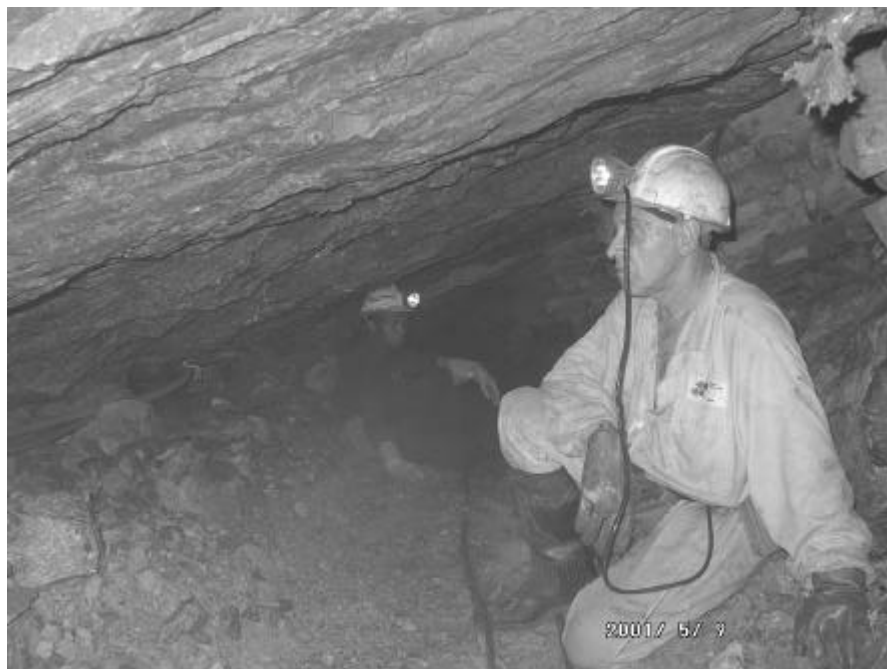


Figure 4.5.3 Photograph of view along the 50-25 E5 gully at Driefontein 5 Shaft

4.5.2.1 Discontinuity properties

Three geological discontinuity (joint) sets were identified and their orientations are shown in Figure 4.5.4. Note that the orientations of set 2 and set 3 joints have the potential to form wedges but no falls of ground (FOGs) were observed. The orientations of the two fracture sets (induced by mining) are shown in Figure 4.5.5. Both of the fracture planes strike parallel to joint set 3 (cf. Figure 4.5.4) and not the face, as would be expected from the stress field. The reason is probably that joint set 3 forms an inherent weakness in the rock mass that is sufficiently close to the direction of the stress field that the fractures preferentially follow the weakness. The dip angle and direction of the steeper fractures appear to confirm this, having the same average dip and dip direction as joint set 3.

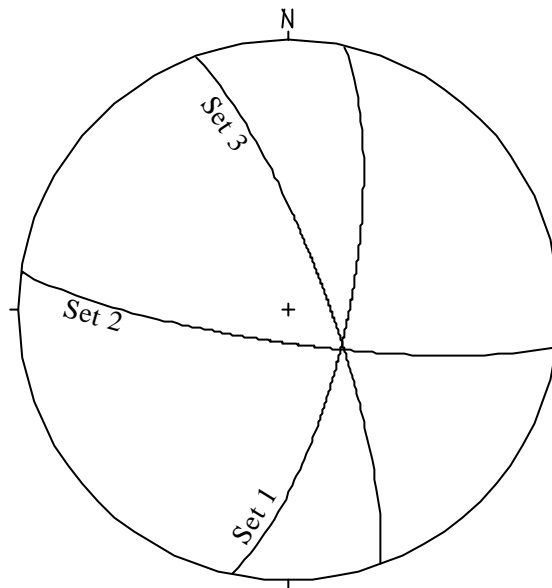


Figure 4.5.4 Equal-area stereo net showing the orientations of the three joint sets observed at Driefontein 5 Shaft

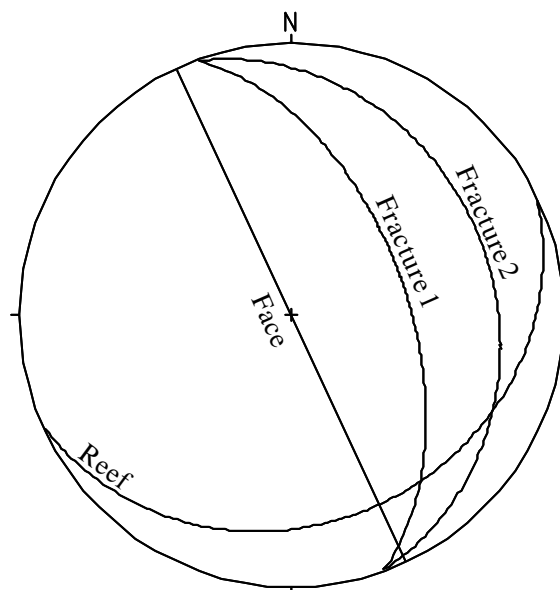


Figure 4.5.5 Equal-area stereo net showing the orientations of fracture sets observed at Driefontein 5 Shaft, compared to the reef dip and face orientation

The geotechnical properties of the discontinuities shown in Table 4.5.1 do not appear to have deteriorated with time. The roughness was measured as the greatest offset from a 0.5 m long straight edge, placed across the surface of the discontinuities. The joint roughness parameter for the classification systems used in the analysis below was determined from these offsets using the method shown in section 3.2.2.1 and Figure 3.2.3.

Table 4.5.1 Discontinuity properties measured at the Driefontein 5 Shaft, 50-25 instrumentation site

Discontinuity type	Persistency	Roughness over 0.5 m	Infill type	Infill thickness	Rock type	Spacing
Set 1	>5 m	7 mm	Quartz	5 mm	Quartz	1.2 m
Set 2	2 m	7 mm	Quartz	3 mm	Quartz	2 m
Set 3	2 m	13 mm	Cal	0.8 mm	Quartz	0.5 m
Fracture 1	1 m	7 mm	-	-	Quartz	
Fracture 2	1 m	6 mm	-	-	Quartz	

4.5.2.2 50-25 E4 gully

The results of surveys performed along the gully at the bottom of panel 50-25 E4 are shown in Table 4.5.2. All surveys were carried out on 5 m lengths in the gullies at the distances indicated from the face position at the time. All the analyses showed that better conditions occurred in this gully 57 m from the face than in the area closest to the face.

Table 4.5.2 Results of geotechnical survey in 50-25 E4 gully at Driefontein 5 Shaft

Distance from face	Window length	RQD	RMR'	Q'	Average spacing of fractures	No. of shallow dipping fractures	No. of steeply dipping fractures
5 m	5 m	5.6%	66	1.22	62 mm	45	36
57 m	5 m	25.4%	71	3.10	114 mm	37	7

4.5.2.3 50-25 E5 gully

Table 4.5.3 shows the results of the surveys performed in the gully at the bottom of panel 50-25 E5. The results of the analyses performed in this gully were similar to those of 50-25 E4, except for the shallow dipping fractures, which showed a systematic increase in numbers, and the RMR', which was unchanged. The photographs in Figure 4.5.6 and Figure 4.5.7 show a typical spacing of fractures as viewed in 'window 1' (5 m from the face) and 'window 3' (60 m from the face), respectively.

Table 4.5.3 Results of geotechnical survey in 50-25 E5 gully at Driefontein 5 Shaft

Distance from face	Span	RQD	RMR'	Q'	Average spacing of fractures	No. of shallow dipping fractures	No. of steeply dipping fractures
5 m	5 m	11.6%	63	1.42	49 mm	33	69
24 m	5 m	14.8%	63	1.81	44 mm	48	65
60 m	5 m	15.4%	63	1.88	42 mm	81	39



Figure 4.5.6 Photograph of steeply dipping fractures in hangingwall of 'window 1', (5 m from face) 50-25 E5 gully at Driefontein 5 Shaft



Figure 4.5.7 Photograph of shallow dipping fractures in 'window 3', (60 m from face) 50-25 E5 gully at Driefontein 5 Shaft

4.5.2.4 50-25 E6 gully

Table 4.5.4 shows the results of the survey performed in the gully at the bottom of panel 50-25 E6. The results of the analyses performed in this gully were similar to those of 50-25 E5, except for the shallow dipping fractures. However, there was a dramatic increase in the number of these fractures between 'window 1' and 'window 3'. There also appeared to be a slight increase in weathering along the discontinuity surfaces between these two windows.

Table 4.5.4 Results of geotechnical survey in 50-25 E6 gully at Driefontein 5 Shaft

Distance from face	Span	RQD	RMR'	Q'	Average spacing of fractures	No. of shallow dipping fractures	No. of steeply dipping fractures
5 m	5 m	6.0%	63	1.22	45 mm	40	70
34 m	5 m	10.4%	63	1.27	38 mm	76	57
62 m	5 m	11.6%	63	1.42	52 mm	72	24

4.5.2.5 50-25 E7 gully

Table 4.5.5 shows the results of the survey performed in the ASG at the bottom of panel 50-25E7. No trends could be established from the results of the analyses, except for the numbers of shallow and steeply dipping fractures. This trend is similar to the other gullies surveyed at Driefontein 5 Shaft.

Table 4.5.5 Results of geotechnical survey in 50-25 E7 gully at Driefontein 5 Shaft

Distance from face	Window length	RQD	RMR'	Q'	Average spacing of fractures	No. of shallow dipping fractures	No. of steeply dipping fractures
8 m	5 m	17.8%	66	2.18	74 mm	15	53
26 m	5 m	13.2%	63	1.61	47 mm	69	37
52 m	5 m	15.0%	66	1.83	60 mm	146	20

4.5.2.6 Discussion

The rock mass ratings did not show any trends that could be associated with time deterioration. Weathering deterioration was observed in only one gully and probably affected only the exposed sections of the discontinuities. However, there was a general increase in shallow dipping fractures with distance from the face (i.e. from 'window1' to 'window3') as shown in Figure 4.5.8. This was accompanied by a decrease in steeply dipping fractures as shown in Figure 4.5.9.

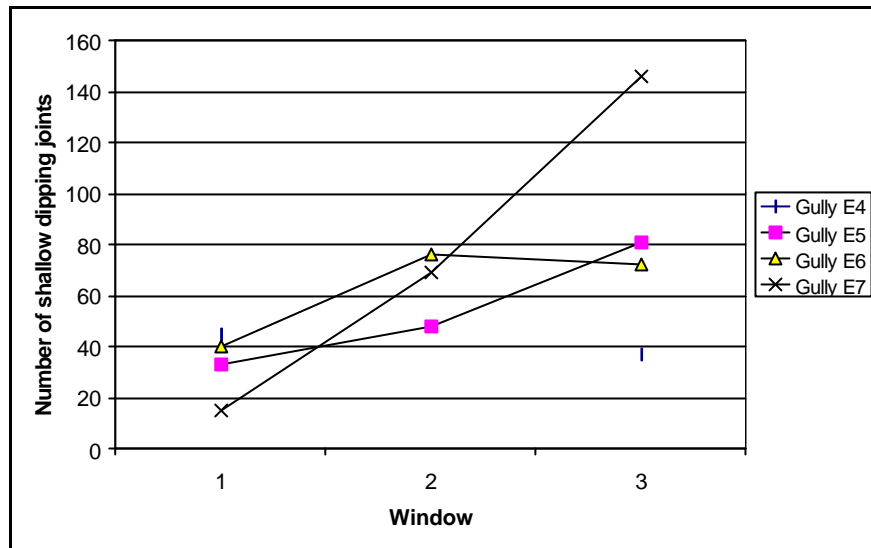


Figure 4.5.8 Frequency of shallow dipping fractures from ‘window 1’ (near the face) to ‘window 3’ (near the centre gully) at Driefontein 5 Shaft

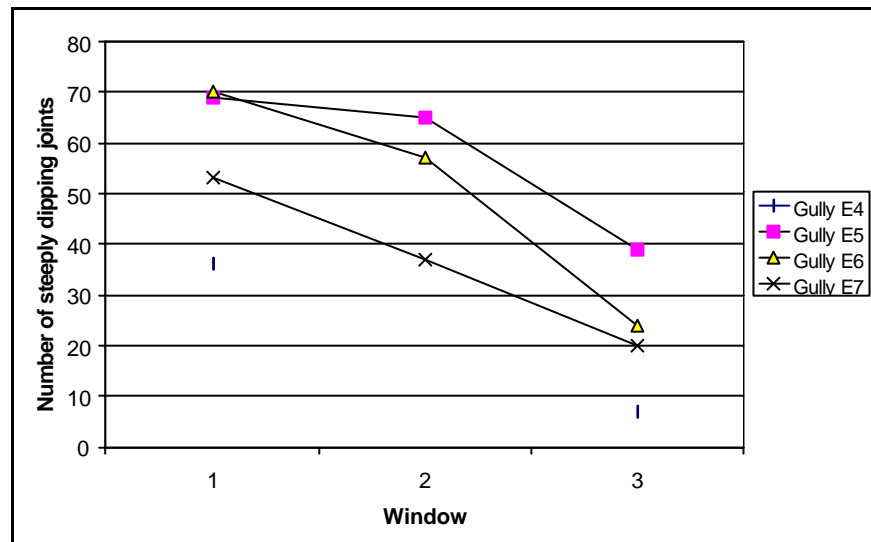


Figure 4.5.9 Frequency of steeply dipping fractures from ‘window 1’ (near the face) to ‘window 3’ (near the centre gully) at Driefontein 5 Shaft

There was also a general increase in the number of shallow dipping fractures down the centre gully towards the abutment at the bottom of the stope, but not a corresponding decrease in the number of steeply dipping fractures.

It is suggested that the phenomenon observed in Figure 4.5.8 and Figure 4.5.9 is not a result of time-dependent deterioration but rather of a change in principal stress direction related to changes in span width. The face was stationary near the centre gully for some months while the western side of the stope was mined out. Once mining progressed on the east side of the stope, the pillar between this stope and the adjacent stope on the east side became narrower. Therefore, the vertical stress increased and the principal stress directions were influenced, thus possibly accounting for the relative increase in steeply dipping fractures, and, decrease in dipping fractures. The influence of the relatively larger span around panel E7 compared to that above panel E4 could account for the trend of increased shallow dipping fractures from panels E4 to E7 along the centre gully.

4.5.3 Ground motion and closure

The PPVs close to the face and far from the face measured in E5 panel were analysed and are plotted in Figure 4.5.10. The panel is situated in the middle portion of the 50-25 stope, which is almost mined to completion. Although the surrounding area is not extensively mined out, the raise line to the east (i.e. 50-26 stope) is mined out and the dip pillar has almost been cut to its final dimensions. The PPVs close to the face were found to be higher than the PPVs recorded in the back area. The difference, however, is not that large and the shape of the curves for both locations is similar. This result is in contrast to that obtained from the comparable VCR site at Mponeng mine, where the difference between the PPVs measured close to the face and far from the face was very significant. The larger uninterrupted dip span of the mining around the monitoring site has possibly influenced the PPVs measured far from the face, although there is little difference between PPV results recorded close to the face at both sites.

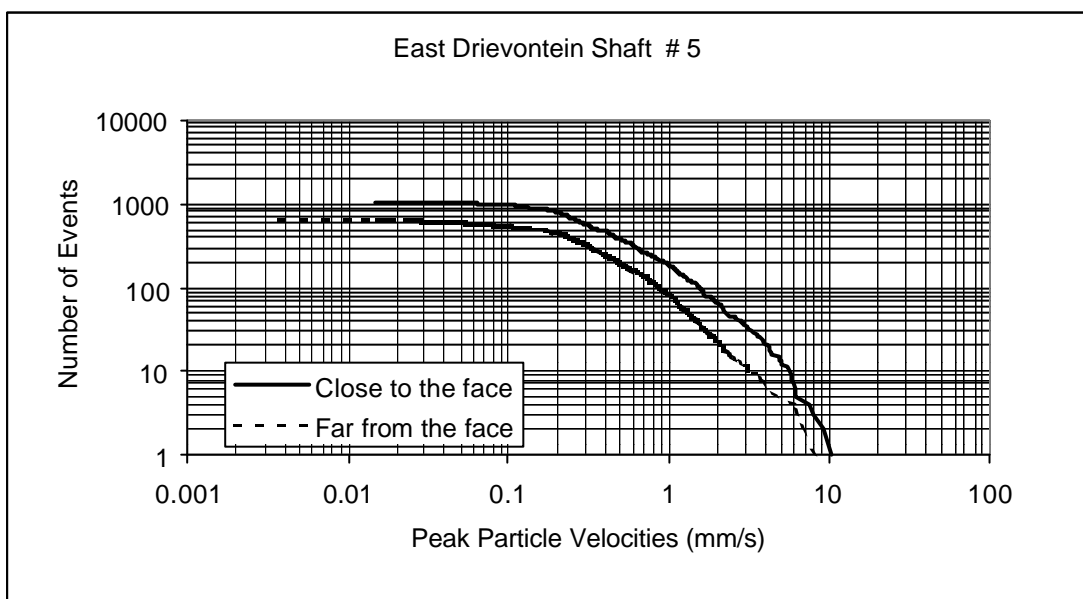


Figure 4.5.10 Peak particle velocities recorded close to the face (upper line) and far from the face (lower line) at Driefontein 5 Shaft

In Figure 4.5.11, both the theoretical and observed dynamic closure are plotted against the PPVs for this site.

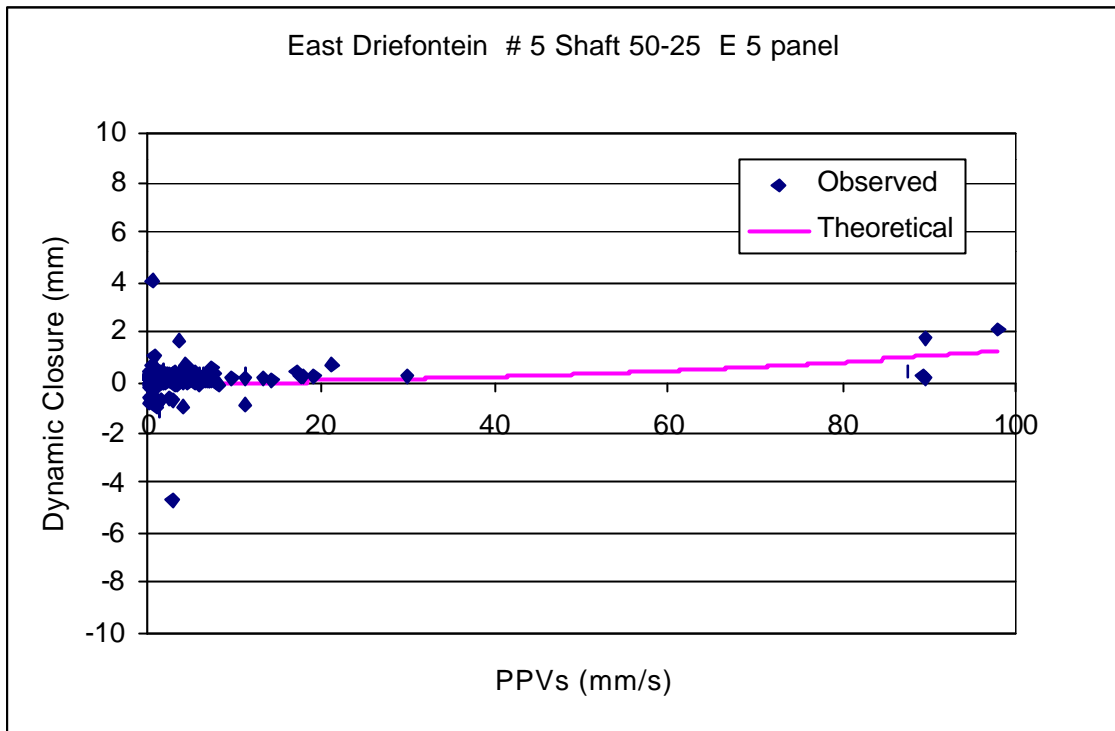


Figure 4.5.11 *Dynamic closure as a function of PPV obtained for 50-25 stope, E5 panel at Driefontein 5 Shaft*

It is clearly indicated in Figure 4.5.11 that the dynamic closure does not increase with an increase in PPVs. A similar effect was obtained for the areas close to the regional support at Tau Tona and Deelkraal mines. It is also interesting to compare these results with the results obtained at Mponeng mine. The main differences between these two sites are: the reef mined (Carbon Leader vs VCR); and the use of backfill at Driefontein 5 Shaft compared to the conventional composite pack support at Mponeng. Other factors, including mining layout, are relatively similar. The dynamic behaviour is, however, different for both sites. The backfilled site at Driefontein (Figure 4.5.11) shows higher resistance to dynamic closure than the unfilled site at Mponeng (Figure 4.4.7).

4.5.4 Numerical modelling

Figure 4.5.12 and Figure 4.5.13 show (for backfill and non-backfill scenarios, respectively) the vertical stress modelled, as a function of monthly face advance, along a transect line that crosses the final extent of the dip stabilising pillar ahead of the monitored E5 panel at Driefontein 5 Shaft. The effect of the backfill is reflected as an almost 10 per cent reduction in pillar stress. For comparative purposes, Figure 4.5.14 summarises the initial and final vertical stress profiles across the dip pillar for backfill and non-backfill cases. Figure 4.5.15 shows (for backfill and non-backfilled cases) the modelled build-up of vertical stress, with increasing face advance, at a fixed point in the pillar ahead of the position that corresponds to the ground motion/closure monitoring station in the E5 panel.

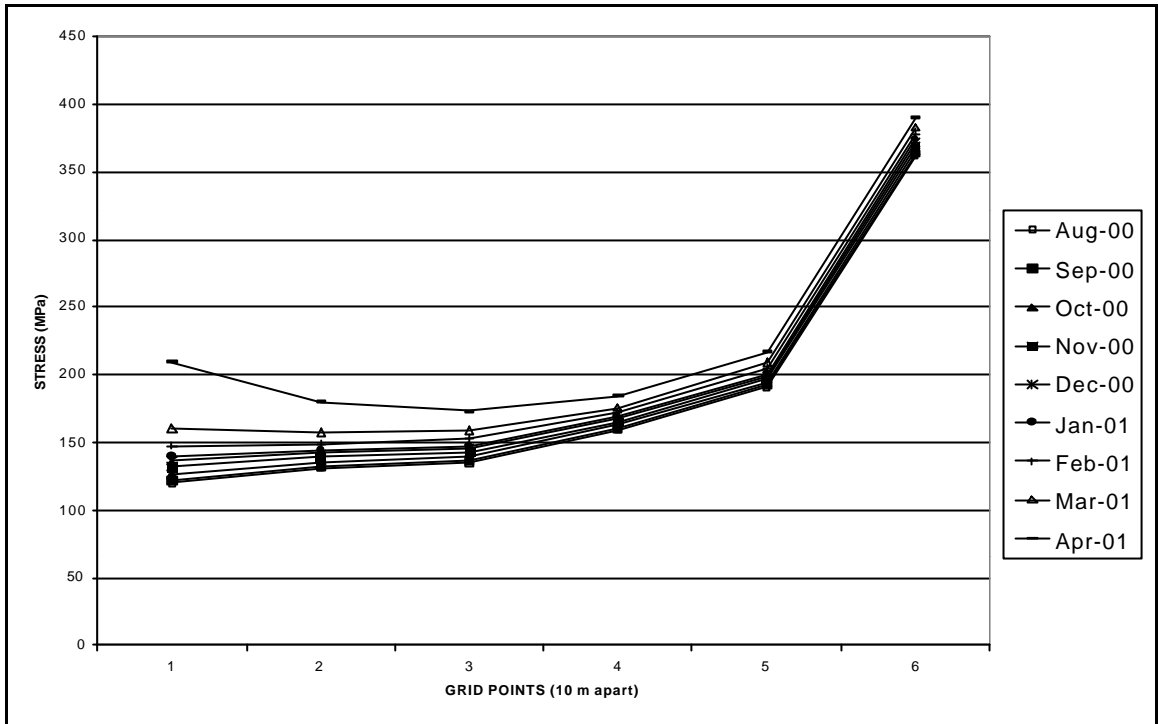


Figure 4.5.12 Modelled vertical stress profiles across dip stabilising pillar with backfill for given monthly face position at Driefontein 5 Shaft

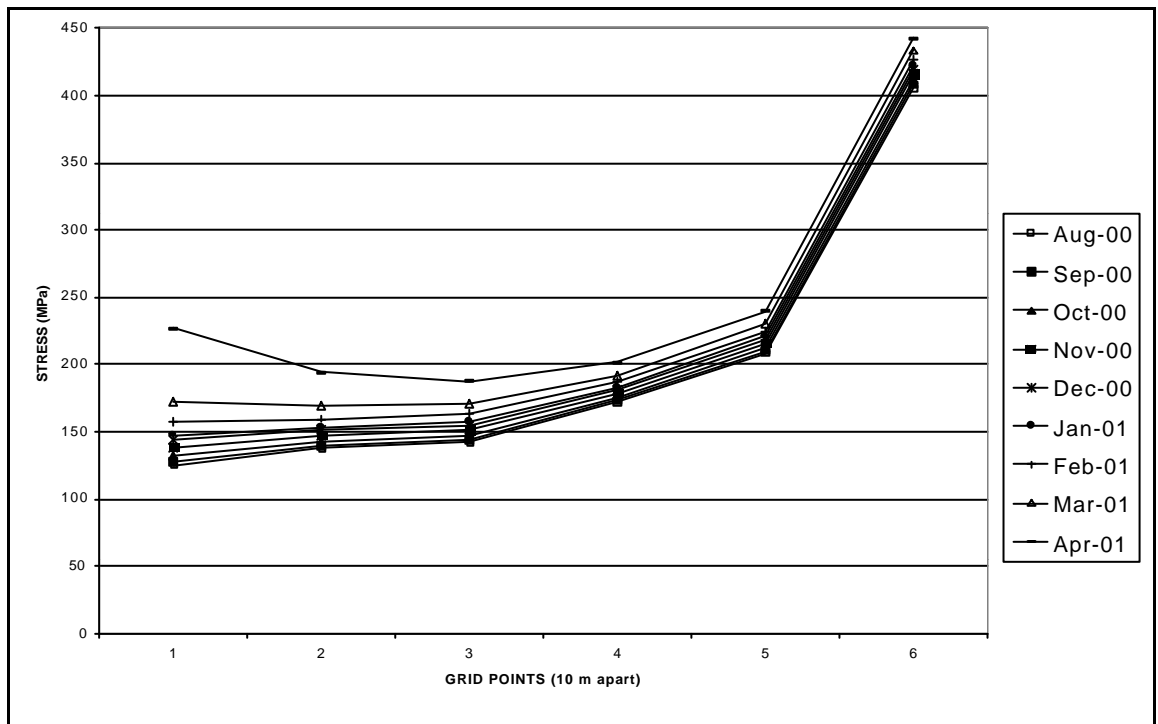


Figure 4.5.13 Modelled vertical stress profiles across dip stabilising pillar without backfill for given monthly face position at Driefontein 5 Shaft

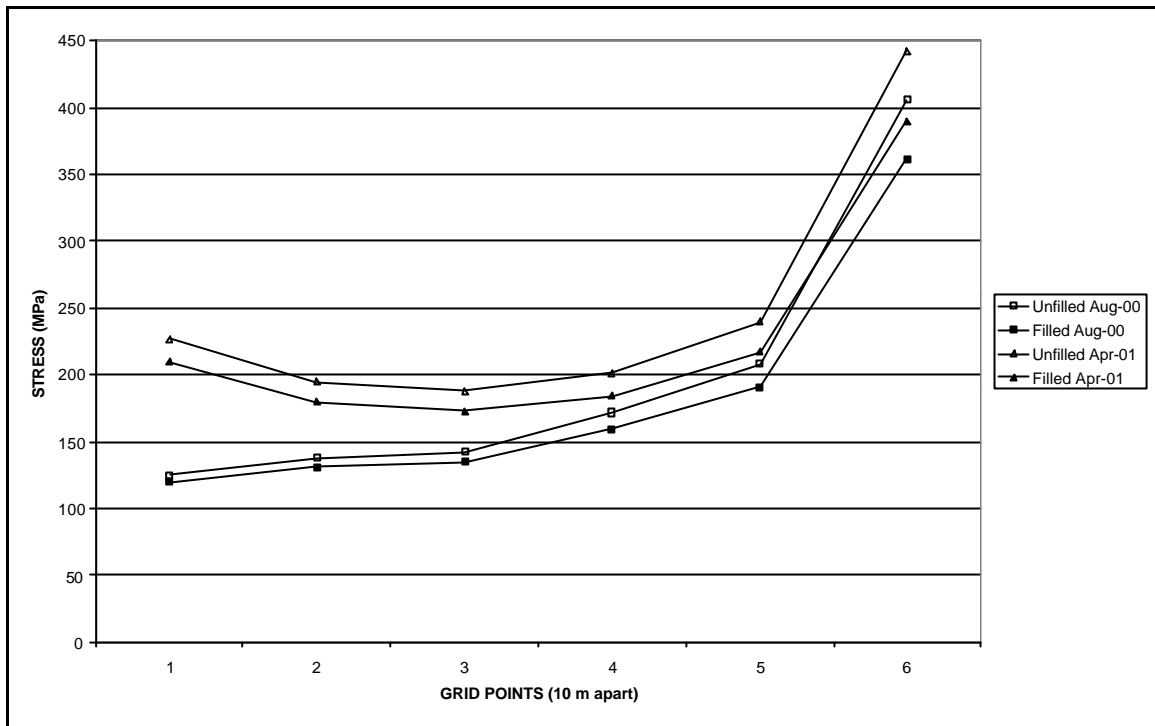


Figure 4.5.14 Comparison of initial and final modelled vertical pillar stress profiles with and without backfill at Driefontein 5 Shaft

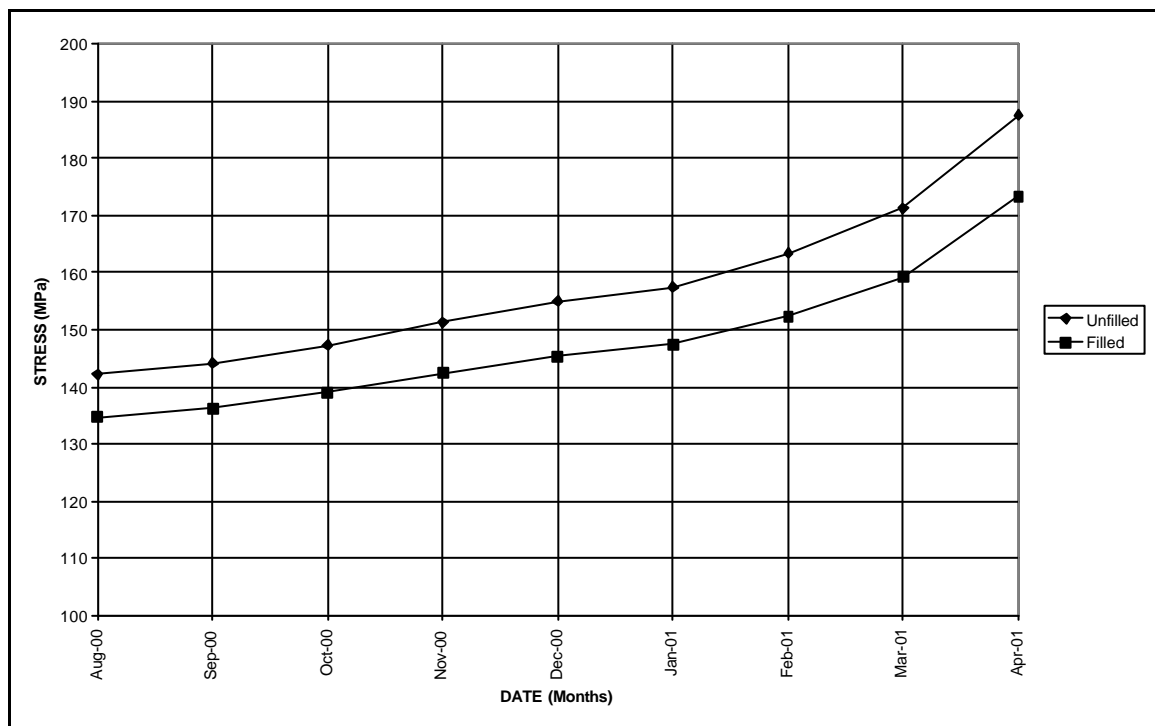


Figure 4.5.15 Modelled vertical pillar stress at fixed location in dip pillar as a function of monthly face advance with and without backfill at Driefontein 5 Shaft

4.5.5 Summary

The key findings from the Driefontein 5 Shaft site can be summarised as:

- Rock mass ratings did not show any trends that could be associated with time deterioration.
- There is a general increase in shallow dipping fractures with distance from the face with a concomitant decrease in steeply dipping fractures, which may be explained by changes in principal stress direction.
- Dynamic closure does not increase with an increase in PPVs, which is a similar effect to that obtained for areas close to the regional support at Tau Tona and Deelkraal mines. In addition, this site shows a higher resistance to dynamic closure than the comparable unfilled site at Mponeng.
- Numerical modelling indicates that the effect of backfill is to reduce dip pillar stress by almost 10 per cent.

4.6 Driefontein 1 Shaft: Special case study

A Ground Motion Monitor (GMM) comprising eight geophones was installed in the 30-31 VCR stope, E9 panel, at Driefontein 1 Shaft. The aim of this monitoring was to analyse the differences in the PPVs on a micro scale. This stope comprises a mini-longwall layout with strike and dip regional pillars and backfill.

4.6.1 Review of installed support

Both conventional and backfill support components are used for in-stope support at this site.

4.6.1.1 Conventional support

Permanent support consists of rows of pipe sticks between which backfill is placed. On the north (i.e. upper) side of the gully, a line of long axis 2.4 mx1.2 m timber packs is placed and 2 m above this a line of 1.2 mx1.2 m packs is placed. There is a one metre gap above this line of packs to the lower edge of the placed backfill. A further line of 1.2 mx1.2 m packs is installed on the south (i.e. lower) side of the gully, adjacent to the upper edge of the placed backfill. Thus only 12 m of the 22 m panel length, i.e. 54 per cent, is backfilled.

4.6.1.2 Backfill

Backfill of a minimum porosity of 42.6 per cent is placed between the rows of pipe sticks. The coefficient of permeability of this backfill is 3.4×10^{-4} , which is lower by a factor of 20 than that achieved by other mines. The hyperbolic 'a' and 'b' parameters are 18.7 and 0.345, respectively. The material is well cut with an above average percentage of fines, with 25 per cent at -38 micron and 8 per cent at -10 micron, which accounts for low porosity and locked-in moisture. A summary of laboratory tests provides the following:

- The above average percentage of -10 micron material in the backfill (average 8 per cent) is producing a backfill that drains poorly.
- The amount of water in the backfill after 9 days was a high 17 per cent at the bottom of the bag, while the middle of the bag produced an acceptable 9 per cent.

- The RD of the placed backfill, measured using the constant-volume technique, indicates that the density of placed backfill after 9 days was on average 1.8.

Poor hanging wall contact, due to an uneven hangingwall and incomplete filling of the bag, resulted in a maximum of 9 m of hangingwall being in contact with the front backfill bags.

4.6.2 Ground motion monitoring

Six of the eight geophones were placed on the hangingwall and two on the footwall. Some of the geophones were placed at the edge of the backfill and the others at mid distance between the backfill and the gully. Figure 4.6.1 illustrates the position of the geophones with respect to the stope face and the gully.

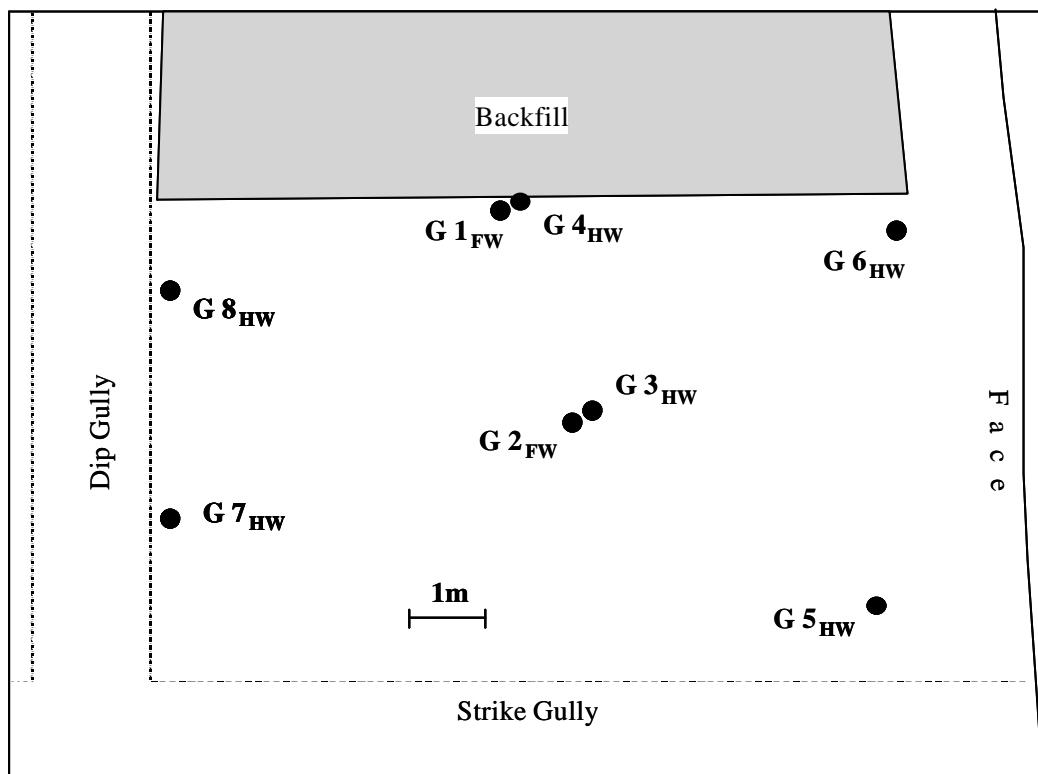


Figure 4.6.1 Schematic of GMM installation at Driefontein 1 Shaft

The peak particle velocities recorded at points on the footwall (G1) and hangingwall (G4) close to the edge of the backfill and on the footwall (G2) and hangingwall (G3) some distance away from the backfill are shown in Figure 4.6.2.

A high degree of similarity in peak particle velocities was recorded on the footwall and the hangingwall near the edge of the consolidated backfill and those recorded several metres away. This indicates that the influence of the consolidated backfill extends some distance into the unfilled portion of the gully edge. The PPVs recorded on the footwall are slightly higher in their middle range, apparently affected by the development of the footwall gully close to the monitoring site.

The peak particle velocities recorded at two points (G5 and G6) on the hangingwall close to the face (in line with the unconsolidated backfill front) and two points (G3 and G4) on the hangingwall further back from the face (in line with consolidated backfill) are compared in Figure 4.6.3.

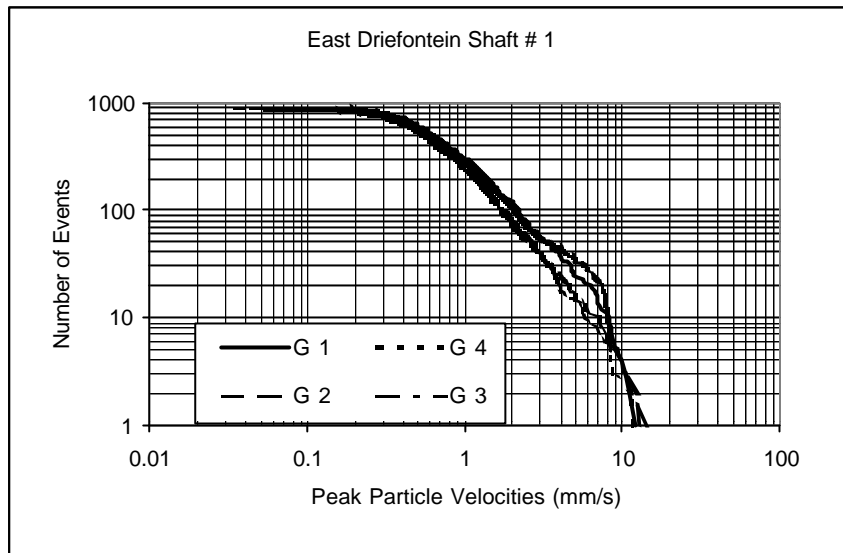


Figure 4.6.2 Peak particle velocities recorded on the footwall (G1) and hangingwall (G4) close to the backfill, and on the footwall (G2) and hangingwall (G3) some distance away from the backfill at Driefontein 1 Shaft

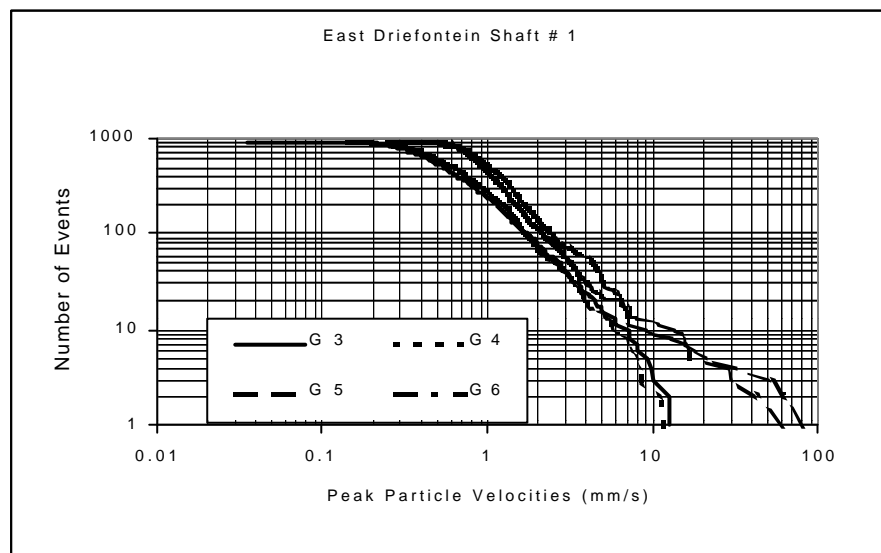


Figure 4.6.3 Peak particle velocities recorded at two points on the hangingwall (G5 and G6) close to the face and two points (G3 and G4) about 6 m further back from the face at Driefontein 1 Shaft

It is clear from the Figure 4.6.3 that the number of seismic events with PPVs up to 10 mm/s is similar for all four positions. However, large PPVs (up to 80 mm/s) are obtained only at the geophones located closer to the face.

4.6.3 Summary

The key findings from the Driefontein 1 Shaft site can be summarised as:

- No trends in fracturing could be established; and
- Higher PPVs are only recorded close to the face.

4.7 Vaal River mines

It was decided that research findings should not be restricted to measurements obtained from site running activities, but also from other backfilled stopes. For this purpose appropriate stopes were visited and the general ground conditions assessed using observational techniques. This additional task is primarily targeted at the mines in the Vaal River Operations (e.g. Great Nologwa and Kopanang). Only two visits were possible and these were both undertaken at the Great Nologwa Mine. The summary of the general information obtained from the mine Rock Mechanics department and the actual underground observations are presented below.

Great Nologwa Mine has a very complex geological structure. No panel is without a minor fault and mining does not proceed for more than 50 m before the next structure is intersected. The majority of the production (~99 per cent) comes from Vaal Reef. The Vaal Reef has an MB3 argillaceous quartzite hangingwall (UCS 175–200 MPa) comprising beams that are a maximum thickness of 1.6 m. In addition, these beams are frequently laminated and comprise layers of 300-400 mm in thickness with shale infilling. The footwall rock is very weak (UCS 170 MPa) resulting in either elongates punching or footwall heave in between support units. The areas of cross-bedding (thickest beam 1.8 m) and the thin laminated bedding (300-400 mm thick) with shale inserts near the reef horizon are also problematic. As wide channel width changes to narrow channel width the hangingwall beams get thicker and, therefore, the ground becomes more stable.

Both visits took place in stope panels in the Moab Khotsong Shaft pillar area, which is being mined by the Great Nologwa mine. This operation is designed to be carried out with backfill at high filling percentages. In general, backfill placement constitutes about 8-10 per cent of the total area mined at Great Nologwa. However, in the Moab Khotsong shaft pillar extraction, the filling percentage is 75 per cent. In other areas backfilling is done at the request of production personnel as ventilation barriers or gully protection ribs. Backfill containment is by a paddock system and the normal filling rate for a paddock is 3-4 hours at 28 tons/hour.

Apart from holding up the back areas no benefits are reported. There are, however, negative reports of backfill making hangingwall conditions worse and of a higher incidence of rockfall accidents in backfilled stopes. In areas of cross-bedding, the backfill (2 to 6 m from the face) does not improve face conditions but it does provide support in the back area. Problems also arise from the associated high stope width, e.g. 1.8 m. The filling rate in high stope widths becomes 10-12 hours instead of the normal 3-4 hours.

It is reported that backfill paddocks of up to 4 m wide are constructed and filled. Consequently, the relevant panel is stopped for 4 to 5 days to allow for the backfilling operation to be completed. The theory being proposed by the rock mechanics department is that during the time that the panel stands, a flat dipping tensile fracture develops at the face and this can lead to a FOG and often a related accident/fatal. Bedding thicknesses are usually 200 mm to 400 mm in the affected areas. Apparently, this failure is also observed when non-backfilled panels are stopped for sweepings. The implication is that this failure happens any time that a panel stands for an extended period of time.

Difficulties were experienced and reported in filling panels in the Moab shaft pillar. On the day of the visit, the closest backfill-to-face distance was around 10 m in one panel. In another panel the backfill was as far as 25 m from the face. On both visits large falls of ground at the panel faces were observed. Falls of ground were observed to be associated with separated bedding planes and mining-induced fracturing. Installing backfill bags too far from the face and poor face area support were thought to be the main reasons for the very poor ground conditions.

5 Discussion

5.1 Hangingwall condition

The results of hangingwall profiling at Tau Tona mine reflect an increased hangingwall roughness, i.e. fallout, nearest to the strike stabilising pillar. Further observations also showed increased fracture intensity and orientation complexity near to the pillar. This indicates the need for an increased areal support capability in this region.

In general, the hangingwall condition in the proximity of good-quality and well-placed backfill was observed to be better than in equivalent unfilled or badly filled sites. No trends were observed in the rock mass ratings at any of the sites that could be associated with time dependant deterioration. However, there was obviously deterioration in the overall hangingwall conditions at Deelkraal 1 Shaft after the installation of the ground motion monitoring instrument as this could not have been installed under the conditions observed during the first rating survey. Many blocks had fallen out of the hangingwall and the remaining blocks appeared to be loose. From the results of the surveys it is concluded that time-dependent deterioration does not involve additional fracturing or extension of existing fractures or joints but the loosening and falling of existing blocks. From these results and observations it appears that pillars orientated perpendicularly to the general fracture orientation could cause greater unravelling of the hangingwall than pillars orientated parallel to these fractures, even in the presence of backfill.

The results of the work performed at the Driefontein 5 Shaft dip-pillar and backfill site indicate, from observed fracturing, that the major principal stress direction changed, as mining progressed on the east side of the stope, from a shallow angle to a steep angle. This change produces a potentially more favourable fracture orientation as the final dip pillar cut-off position is reached. In the course of these investigations there was no evidence that the 'regional dip pillar' type of layout leads to a deterioration in hangingwall conditions in the latter stages of pillar cutting. In the sequential grid and closely spaced pillar breast mining layouts, the shallow dipping fractures associated with the early mining of the raises/centre gullies result in less stable fracture orientations than close to the regional dip pillars. Backfill and effective roofbolting in gullies would alleviate the problems associated with the shallow dipping fractures. Backfill close to face with good working area support as close as possible to the face is also required. However, the increased likelihood of seismicity close to those pillars needs to be catered for in the support design.

5.2 Ground motion and closure

In the course of this project PPVs were analysed in a similar way for backfilled stopes on three different reefs, i.e. Vaal Reef, Carbon Leader and VCR and for different mining layout geometries, i.e. strike stabilising pillars, dip pillars and scattered mining. This allows for a comparison by of the influence of mining environment on the existing support. The attenuation of PPVs with distance from the face is used as an indicator of the amount of support provided the backfill.

The ratio of the PPVs recorded close to the face and far from the face for backfilled stopes at Kopanang, Tau Tona and Deelkraal mines is shown in Figure 5.2.1. This ratio gives an indication of the reduction (i.e. absorption effect) of PPVs that occurs further back from the face due to the presence of backfill.

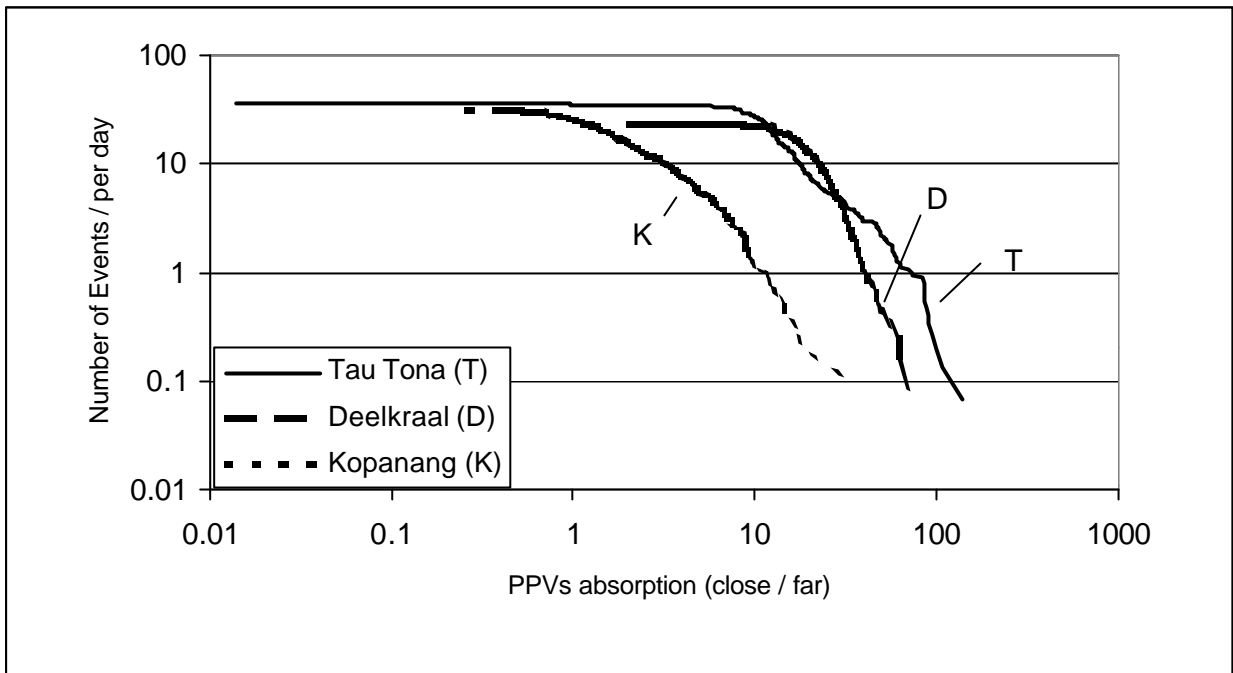


Figure 5.2.1 *Ratio of the PPVs recorded close to the face and far from the face for backfilled panels at Tau Tona, Kopanang and Deelkraal gold mines*

PPVs generated by the ‘face-effect’ microseismic events associated with face advance (Figure 5.2.2) gradually increase with increasing perpendicular distance from the strike stabilising pillar (Figure 5.2.3). A similar trend is also observed for the stronger PPVs analysed for the entire period of monitoring. An increase in pillar stress, which results from face advance, is indicated by the increase in ‘pillar-effect’ microseismicity recorded further back from the face in panels adjacent to the strike stabilising pillar (Figure 5.2.2 and Figure 5.2.3).

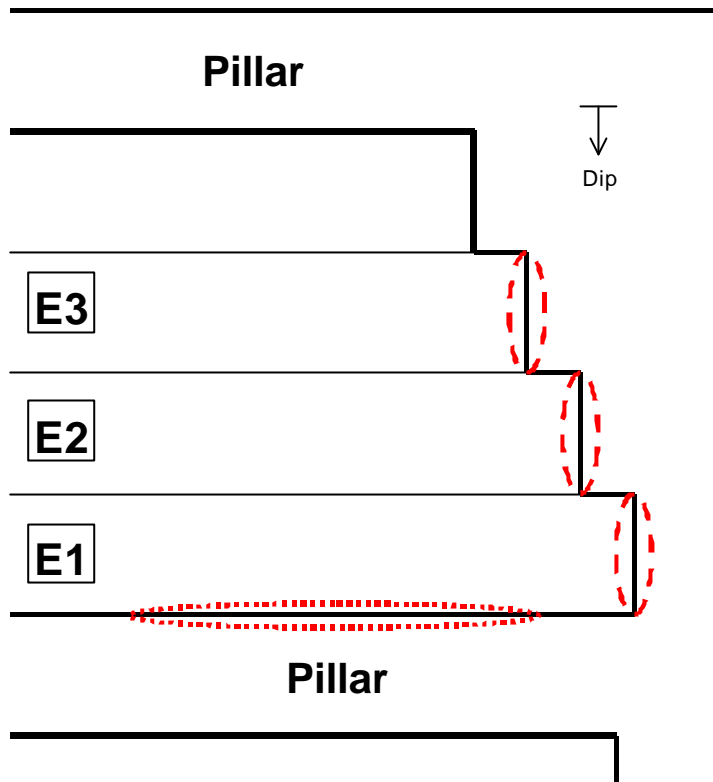


Figure 5.2.2 Schematic of zones of 'face-effect' microseismicity (long-dash ellipses) and 'pillar-effect' microseismicity (short-dash ellipse) affecting PPVs

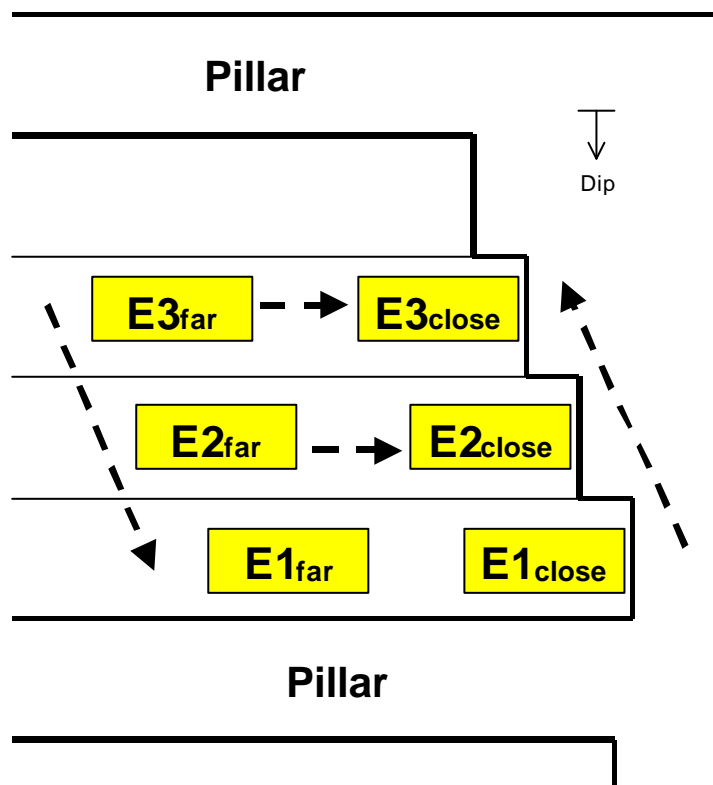


Figure 5.2.3 Schematic of relative magnitude (dashed arrow = increase) of PPVs as a function of proximity to face and strike stabilising pillar at Tau Tona mine

To summarise, the following was found:

- The PPVs increase with increasing perpendicular distance from the pillar (cf. Figure 5.2.3).
- The PPVs recorded close to the pillar increase with the increase in pillar stress associated with the face advance (cf. Figure 5.2.3).
- The reduction in PPVs from the face to the back area varies for different geotechnical areas.

Results from the closure rate measurements indicate that the amount of stope closure increases with increasing perpendicular distance from the pillar and that the rate of closure is greater closer to the face than further back from the face. Therefore, support (backfill and/or conventional units) installed close to the pillar will experience less closure than support that is installed further from the pillar. In addition, support (backfill and/or conventional units) will, in the early stages after installation (i.e. close to the face), experience higher closure rates than in the later stages (i.e. far from the face).

The ground motion results from Tau Tona, Deelkraal and Driefontein 5 Shaft show that the regional support (i.e. pillars and well consolidated backfill) dramatically influences the level of dynamic closure (associated with the PPVs) in the adjacent region. The regional support has the effect of reducing the dynamic closure to a very low value, effectively shielding the support installed in the adjacent region from the dynamic effects of the events generating the PPVs. Conversely, the areas further away from the regional support experience a systematically increasing dynamic closure with increasing PPVs. These effects have implications for the design of in-panel support in the two areas.

The zone of influence of the strike stabilising pillars is seen (from PPV, closure and fracture mapping) to only effectively extend into the closest adjacent panel, whether up-dip or down-dip of the pillar. For practical purposes this zone can be considered to be restricted to the region traditionally occupied by the relevant strike gully and its associated gully support units. However, a review of current industry practice (section 2.5) indicates that, as a general rule, no special design strategies are applied to the selection of support systems for this region.

5.3 Vaal River

The debate continues regarding the influence of backfill on the stability of the face area hangingwall in certain portions of the Vaal River mining district, with opinion divided as to a direct negative effect, indirect negative effect or benign effect. It was not possible during the course of this project to obtain direct scientific evidence as to the actual mechanism or reasons for what prevails in this situation.

One viewpoint is that the presence of backfill in specific portions of the Vaal Reef mining horizon, where thinly laminated hangingwall beams exist, creates detrimental hangingwall conditions and therefore increased falls-of-ground and hazard potential (Macfarlane, Laas and Spearing, 1998; and Laas, 1993). It was not possible at the available sites to either categorically prove or disprove this theory. However, an equally compelling explanation has come to light.

Observations made in certain backfilled panels visited support the contention that different fracture orientations and frequencies cause worse hangingwall conditions than in unfilled panels. These observations must, however, be qualified by the additional observation that poor quality backfill placement, i.e. gaps between adjacent backfill ribs, gaps between backfill and hangingwall and excessive fill-to-face distances, was also invariably evident. As a result, the

effective support in backfilled panels was in all likelihood inadequate and inferior to that in the conventionally supported panels. It must be pointed out that this observation is not backed up by rigorous scientific research. It is, however, consistent with comments made by mine personnel, wherein 'backfilled' panels stand for extended periods of time (e.g. minimum of four or five days) whilst backfill placement is undertaken to catch up with the advanced face position. During these periods it is also likely that insufficient support is in place ahead of the backfill and time-dependent failure of the laminated hangingwall takes place, resulting in the increased falls-of-ground frequently associated with backfilled panels in this region. A similar outcome is noted to occur in conventionally supported panels when these also have cause to stand for extended periods. However, this situation arises less frequently because conventional support installation is more likely to keep pace with face advance.

It is, therefore, believed that the main issue is one of maintaining and managing the correct placement of backfill within a realistic mining cycle.

If the issue of backfilling in the Vaal River region is still considered to be of concern and relevance, it is suggested that a research project specifically dedicated to this purpose be initiated. Such a project will have the clear and explicit intention of instrumenting, monitoring and fully evaluating appropriate underground sites for the clarification of the situation. A number of such sites were not available during the investigation at the Vaal River operations.

5.4 Significance of findings for support design

It is anticipated that support units in the vicinity of strike stabilising pillars will be required to withstand significantly less dynamic and quasi-static closure than units in panels further away from the pillars. Nonetheless, an increase in microseismicity will be encountered in the vicinity of strike pillars because of the increase in pillar stress that results from face advance. Also, areas close to strike pillars, particularly gullies, will have to sustain the cumulative effects of nearby events emanating from the pillar for the lifetime of the gullies. This influence does not apply to the more distant stope panels, which are only subjected to nearby events for a short period when they comprise the working area of the stope. Thereafter, they are in the back area and more distant from the damaging source area of the seismic events. Increased areal coverage will be required to counter the more intensely fractured hangingwall and footwall close to the pillars. This may necessitate the use of headboards and footpads on elongate type units.

There is no evidence that dip pillar regional support layouts require a different in-panel support strategy as the face positions approach the pillar cut-off position, apart from the increased likelihood of face related seismicity that must be taken into account. However, the presence of backfill in these panels does provide the opportunity for a modified face support design, different from that determined for unfilled situations. Although starting from a lower base level, the attenuation of PPVs is greater in backfilled panels than equivalent unfilled panels. In the absence of an increased understanding of the zone of influence of backfill as local support, the work of Daehnke *et al.* (1999) is identified as providing a theoretical approach to this aspect when required for the design of face area support ahead of backfill.

5.5 Methodology

Although certain trends have been identified in factors influencing local support design in the vicinity of regional support, no generic criterion can be provided that will apply to situations other than those sites studied. It is, therefore, proposed that rather than an attempt to impose a

criterion, a methodology be developed that will guide towards the correct design of local support for each situation encountered.

Taking into account the findings of the current project work, the methodology shown in Figure 5.5.1 is proposed as the most meaningful solution to the design of local support, given the diverse conditions and configurations that exist. This methodology involves assessments of existing or anticipated stress build-up in pillars (or well consolidated backfill) adjacent to panels, closure profiles and hangingwall condition near to and away from pillars, and ground motion or seismic history.

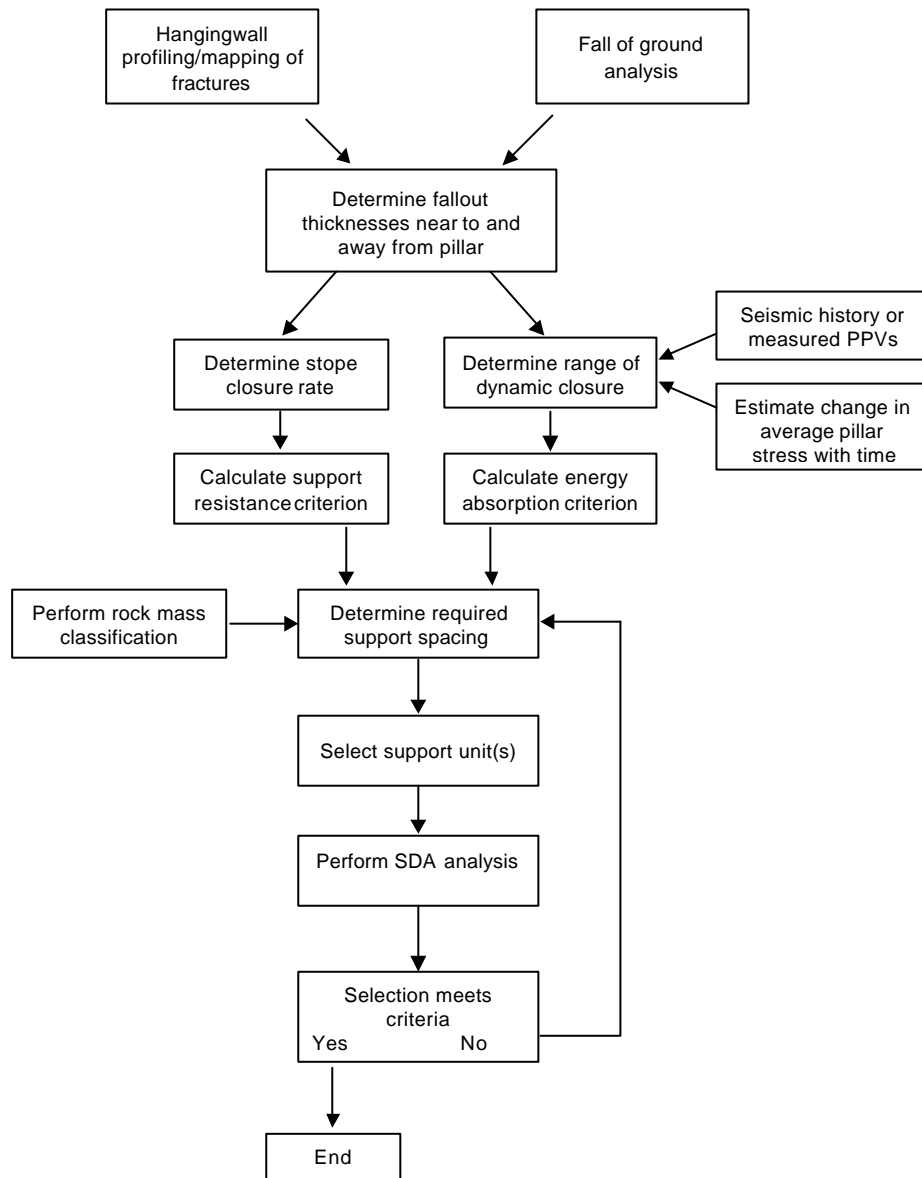


Figure 5.5.1 Methodology for design of local support in vicinity of regional support

5.6 Platinum mines

The findings from the gold mine-based studies do not have direct relevance for the platinum mines since the equivalent conditions to those monitored do not occur. In as much as dip pillar layouts are used on the platinum mines, the current work has not indicated the need for different support systems or strategies on approaching the pillar cut-off position.

6 Technology transfer

A series of technology transfer workshops are to be held, in consultation with SIMRAC, at appropriate venues after approval of this final report has been granted.

7 Conclusions

The following conclusions can be drawn from this work:

- The issue of backfill influence on hangingwall conditions in the Vaal River mining region could not be adequately resolved. No direct evidence was observed of backfill creating worse hangingwall conditions; instead it was observed that poor backfill placement was associated with the less favourable hangingwall conditions.
- Generally, well placed backfill improves conditions in face areas if it is kept close to the face and conventionally designed working area support that fits in well with the backfilling/mining cycle is implemented. Conversely, quality is not assured if backfill is not well placed. Also large fill-to-face distances are the result of regular filling not taking place and these distances, together with inadequate working area support, lead to deteriorating and unsafe conditions. Combinations of the above poor practices are implicit in the findings of Squelch and Görtunca (1991) who state that accident rates are higher in backfill stopes than in conventionally supported stopes if less than 60 per cent of the stope is backfilled. To achieve the benefits of backfill, strict adherence to a well-established set of standards for both backfilling and working area support must be ensured.
- Dynamic closure, resulting from events generating PPVs, is reduced in the vicinity of regional support (pillars and well consolidated backfill).
- Support units in the vicinity of strike stabilising pillars will be required to withstand less dynamic and quasi-static closure than units in panels further away from the pillars.
- A greater relative increase in peak particle velocities (PPVs) will be encountered in the vicinity of strike pillars than in areas closer to the middle of the stope.
- Areas close to strike pillars, particularly gullies, have to sustain the cumulative effects of nearby events emanating from the pillar for the lifetime of the gullies.
- Conditions in gullies adjacent to backfilled panels are generally considered to be at least as good as if not better than those where conventional support is used, particularly under rockburst conditions. The question of whether to backfill up to the gully edge with or without internal reinforcing is currently being researched and did not form part of this project.
- The PPVs recorded close to the face adjacent to the recently placed backfill are higher in all monitored panels than the PPVs recorded further back in the areas where the backfill is already consolidated and provides a better support. Similar behaviour was found for all backfill sites at Kopanang, Tau Tona and Deelkraal mines. Any differences from site to site can be explained by variations in the local conditions.

- It is not possible to provide a generic design criterion for local area support in the vicinity of regional support, because of the variable nature of conditions that exist and the limited nature of the research study. It is, however, relevant to apply a methodology to the process of determining the criterion for each situation.
- A methodology is proposed for the design of local area support in the vicinity of regional support.

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Appendix A Numerical modelling input parameters for Minsim2000

Backfill Hyperbolic Parameters

Mine	Backfill Type	Porosity	a	b	Fill Width
Tautona	CCT	44.3%	21.9	0.3278	1m
Deelkraal	CCT	43%	25.9269	0.3237	1m
EastDriefontein #5	CCT	43.7%	19.4292	0.3466	1m
Moponeng	-	-	-	-	-

Geometric and Rockmass Parameters

Mine	Young's Modulus E (GPa)	Poisson's Ratio	Rock Density (kg/m ³)	Stoping width (m)	k-Ratio	Sheet Size	Grid Size (m)
Tautona	70	0.2	2700	1	0.5	128x128	10m
Deelkraal	70	0.2	2700	1	0.5	128x128	10m
EastDriefontein #5	70	0.2	2700	1	0.5	128x128	5m
Moponeng	70	0.2	2700	1	0.5	128x128	5m

Rockmass Parameters	
Young's Modulus E (GPa)	70
Poisson's Ratio	0.2
Rock Density (kg/m ³)	2700
k-Ratio	0.5
Backfill Parameters	
Sheet Parameters	
Used the same parameters derived from Deelkraal Test Results, Quadratic	<i>Average Stoping Width</i>
	<i>Fill Width</i>
Tautona	1.0
Deelkraal	1.0
Moponeng	1.0
Geometry	
Sheet Size	128 X 128
Grid Size (m)	10