

# SIMRAC

## Final Project Report

Title: DEVELOP CRITERIA FOR DESIGNING MINING LAYOUTS AT DEPTH SOP AS TO REDUCE SEISMICITY AND ENHANCE WORKER SAFETY: WORK COVERED DURING 1993 AND 1994

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## EXECUTIVE SUMMARY

Two years of the original three year contract have been completed and considerable progress has been made in the deep level layout design criteria project.

The progress in research on bracket pillars has been encouraging. Literature and industry surveys were invaluable in establishing the presently used methods and experience in designing bracket pillars. Examples of successful and unsuccessful bracket pillars adjacent to dykes and faults were collected for further analysis. A data base for dykes and faults has been established in order to determine the relationship between their behaviour and their characteristic properties. Unfortunately there are no infallible criteria that classify a dyke or fault as stable or unstable. Two-dimensional numerical modelling of stiff rock, comparable to dyke material, surrounded by less stiff rock demonstrated a concentration of stress in the stiffer material and higher ESS values and larger ESS lobes at the interface between the two rock types compared with interfaces with similar rock types on both sides. Three-dimensional modelling was used to obtain seismic moments, which were compared with the actual seismic moment obtained from seismic records during mining of the area. The modelled seismic moments were reduced by any actual seismicity which occurred in a previous mining step. In spite of this reduction the modelled values over estimated the actual seismic moment suggesting that the numerical modelling is conservative and is thus a valid tool to estimate the seismic risk of a particular layout. Seismic records have been used to determine the potential hazard of structures with regard to mining in their vicinity. Such a method has demonstrated the differences in behaviour of various geological structures. Seismicity does not always increase when mining is the same distance from a fault or dyke. Information from mining in the vicinity of a particular feature may be valuable in mining near that feature in another area and may assist in designing bracket pillars.

In the area of stabilising pillar research there has been progress in understanding the rationale for the original design of the pillars. Accident statistics show a static trend and a decreasing trend with respect to fatalities and casualties since the introduction of stabilising pillars on those mines using this type of regional support. A survey of the value of gold presently locked up in these pillars, the adherence of the present pillars to the design criteria in terms of average pillar stress, energy release rates and extraction ratios has been completed. In many instances the pillar widths were below the design width and average pillar stresses modelled were above the maximum design value. Two stabilising pillar sites have been monitored with portable seismic systems (PSS) and the seismicity of the two sites has shown different characteristics as a result of the different ages of the two pillars. An instrumentation program has been devised to assess closures, fracturing, stresses and dilation associated with the pillars. Stress measurements below a stope between two stabilising pillars suggest that regeneration of stress occurs probably as a result of complete closure in the back area of stopes.

A 6m x 6 m concrete block has been installed underground to monitor its behaviour under high stress and closure conditions. To date limited mining and therefore closure has occurred in its vicinity as a result of slow mining advances. The block has been instrumented with hydraulic stress meters and closure meters inside the block and closure-ride stations surrounding it. In order to obtain the parameters for inelastic numerical modelling of concrete, a triaxial cell has been built to accommodate the aggregate size that may be necessary for stiff concrete. The cell has been used to conduct a number of tests which have produced valuable results.

Suitable parameters have been obtained for inclusion in a critical state model to simulate the behaviour of different backfill materials using FLAC.

In the initial time period an evaluation of the VOLSIM code was completed which highlighted some of the advantages and disadvantages of this numerical modelling scheme. A pre and post

processor was developed and time was spent introducing the code to the industry in a workshop. The code has some features which may be used in regional modelling to simulate inelastic behaviour of the rock, which is critical to understanding the rock mass behaviour better.

Significant progress has been made in understanding the caving mechanisms in back area caving and in quantifying caving behaviour. This work has been supported by numerical modelling, and benefits of such a mining method have been determined. It has been possible to directly measure the stresses generated in the back area of caved stopes.

A number of back analysis studies have been undertaken to assess the usefulness of ERR and ESS. It is evident that geology plays an important role in determining the appropriate technique to be used. The usefulness of incremental area ESS and ubiquitous joint ESS has been demonstrated.

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## DEEP MINE LAYOUT DESIGN CRITERIA

### INTRODUCTION

During 1994 CSIR Mining Technology was invited to make some changes to the scope and completion times of the work being carried out in the GAP034 project. Some of the milestone dates set for delivering outputs were over ambitious, while certain extensions to the work programme were suggested by GAPREAG members. The project was rewritten to cover a new three year cycle commencing at the beginning of 1995. A request was made by the GAPREAG committee to formally conclude the first two years of work with a final report on the progress during that time period.

### BRACKET PILLAR DESIGN

Bracket pillars are widely used in the South African gold mining industry to act as regional support and thus control the seismicity and associated rockburst problem in highly stressed environments. Bracket pillars are unmined ground left adjacent to a fault or dyke in an attempt to reduce seismic slip and thereby minimise the rockburst potential. However, there are no clear guidelines at present available to the mining industry for the successful design of these pillars. Research in this area during the last two years has been aimed at improving the criteria used in the design of bracket pillars. Areas of work include a literature review of strategies used when mining in the vicinity of geological structures, such as faults and dykes; an industry wide survey of current procedures and practices for such mining; the establishment of a fault and dyke database to be used in determining the parameters having the strongest influence on the hazard potential of the structure; numerical modelling and back analysis of seismicity associated with mining in the vicinity of a number of geological structures.

It is well known that there is an increased risk when mining in the vicinity of some dyke or fault structures. A number of workers have highlighted the relationship between underground damage and its proximity to geological structures as well as the increase in both the total seismic energy and the number of damaging events.

Criteria currently used in the design of bracket pillars include excess shear stress (ESS) and the stress to strength ratio. ESS is generally used for determining the width of a bracket pillar required to stabilise a fault. The bracket pillar width is chosen so that the positive ESS lobes are fragmented or minimised. The stress to strength ratio refers to the ratio between the maximum principal stress and the uniaxial compressive strength of the dyke material. In this case the mining layout is chosen so that this ratio remains below unity.

The progress in this part of the project is discussed in five sections, namely:- A literature review; an industry survey; geological structure classification; numerical modelling and seismic back analyses and characterisation.

#### Literature review

The current criteria used for designing bracket pillars have been documented. These are excess shear stress (ESS) - in which the pillar is designed such that positive ESS lobes are fragmented or reduced in extent; and the stress to strength ratio in which the uniaxial compressive strength is compared to the maximum principal stresses. The bracket pillar is designed so that the ratio between the stress and the strength is less than unity.

Other strategies discussed included variations in mining layouts such as the use of the 30° rule; narrow spans and mining on retreat.



Workers also documented some of the implications that geological structures have on mining in their vicinity such as the increase in the number of damaging events and the increase in rock damage in the immediate vicinity of the structures.

Reports in the literature claim that the potential hazard of a fault depends on a critical loss width and whether there has been extensive stripping or mining close to the fault plane. Faults that are planar, extensive and regular, with well defined contacts and a tendency to dilate have the potential to produce large seismic events. Such events are likely to generate low stress drops that are potentially less damaging. In general it appears that the size of a seismic event associated with a fault is proportional to the throw of the fault. Faults with throws which have resulted in rock types of different strengths against one another, or faults with intruded dykes, are also more hazardous.

Similarly, it is claimed that the degree of hazardousness of a dyke depends on the strain energy density, the width of the dyke pillar and the magnitude and orientation of the external stresses acting on the dyke. The dip, the dykes geometry, the degree of decomposition and the uniaxial compressive strength are also reported as important parameters controlling the hazard associated with a dyke. Large, fresh dykes, oriented parallel to dip, with unsheared contacts were identified as being most likely to be associated with rockbursts. In addition strong, brittle, well jointed siliceous rocks show preferential event concentration to weak argillaceous rocks.

Reports of numerical modelling discussed in the literature review covered the following three broad areas:-

- 1) Modelling of bracket pillars and their stability.
- 2) Modelling of events associated with geological structures.
- 3) Modelling of alternative strategies for mining in the vicinity of geological structures.

A number of papers included discussions on the likely mechanisms of failure for both faults and dykes. In general these explanations were related to two concepts. Firstly that of disturbing a pre-mining in-situ stress state, in which relatively small mining-induced stresses are superimposed on the existing stress state, resulting in violent failure within the rockmass. The second concept discussed is that of the storing of stress by dykes with significantly different elastic properties to those of the host rock. The dyke may then be subjected to stress levels which result in failure of the dyke material. Alternatively mining in the vicinity of the dyke structure may then cause relaxation of these dyke stresses which can initiate failure in the surrounding rockmass.

A number of workers have documented their experiences with designing bracket pillars adjacent to geological structures. The reports covered in the literature review involved bracket pillar widths of between 10 and 40 m. The majority of these designs were successful with a reduction in rockburst damage occurring after the implementation of the pillars. The cases documenting unsuccessful designs involved the occurrence of very large events after very limited mining in the area.

The literature review was useful in identifying:-

- i) criteria that are currently being used in designing bracket pillars;
- ii) general procedures for mining in the vicinity of geologically hazardous structures;
- iii) results of numerical modelling of bracket pillars and of geological structures;
- iv) the current state on knowledge on factors controlling the hazard potential of a geological structure;

### Industry survey

The industry survey was carried out with a number of objectives:-

- 1) Determine criteria currently used in designing bracket pillars
- 2) Determine where these criteria were successful and where not
- 3) Identify cases to be used for detailed back analyses and numerical modelling
- 4) Determine other successful and unsuccessful strategies for mining in the vicinity of geologically hazardous structures. Information from different areas will be combined.

Excess shear stress is the most commonly used criteria for designing mining layouts in the vicinity of fault and dyke structures. Generally zero cohesion and a 30° friction angle are used in the modelling. Incremental ESS is also used - i.e. the increase in positive ESS from one mining step to the next.

In addition, the bracket pillar width may be chosen based on the distance at which the average pillar stress (APS) falls below a pre-set value. Other criteria used to decide on the width of the pillar are based on factors such as:-

- i) The distance at which there is a marked increase in the seismicity level
- ii) The distance at which ground conditions deteriorate dramatically
- iii) Financial constraints
- iv) The width of the dyke - in that the total pillar width is the important criterion

In a number of cases, step-on or en-echelon pillars are used in a situation when mining approaches a geological structure at an angle greater than 0°. This results in a saw tooth-like pillar against the structure and although this layout tends to reduce the likelihood of the structure failing, it generally results in a deterioration of local ground conditions.

In the Klerksdorp region bracket pillars are designed using the UCS, age, width, dip, thickness, strength, jointing and strain energy density. If a dyke is known to be burst-prone, then a thin skin (3-5 m) will be left around the dyke in order to stop fragmentation into the stope and to keep people from being within the most dangerous area, 3 m from the dyke contact. These pillars are also used to stop faults from opening up, particularly on the weak side of a fault.

Factors mentioned which were used to decide whether or not a bracket pillar should be left include:-

- i) seismic history
- ii) ore grades
- iii) structure orientation and width

Unconventional bracket pillar shapes are used adjacent to shallow dipping structures where the overlap of the structure over the stopes is large. These include long thin strips left perpendicular to the dyke or fault.

Numerical modelling is difficult in areas with non-tabular deposits. Structures with irregular geometry's such as bulging dykes are also difficult to model. Frequently, bracket pillars are reported to be particularly unsuccessful against structures that have previously been stripped against or mined into. In areas with very limited spans, bracket pillars are not practical and instead the support density is increased.

As a result of the inaccuracy of regional seismic networks and the fact that seismic events often plot at the intersection of two structures it is not always possible to assign particular events to a specific structure and thereby identify the active structure. This is compounded by the fact that in

general there are no observable displacements on structures after seismic events in the Klerksdorp region.

A number of reasons have been identified to explain why bracket pillars have failed, namely:

- i) Pillar is not symmetrical about the structure
- ii) The mining history in the vicinity of the structure
- iii) Bracket pillars within pre-existing remnants adjacent to a geological structure are particularly unpredictable in their behaviour.
- iv) If a structure with an irregular shape is treated as if it had a regular, planar nature then the way that it behaves can be unpredictable.
- v) Areas in which dykes and faults intersect one another can be particularly hazardous.
- vi) The behaviour of combined fault dyke structures is not consistent with that obtained from numerical modelling of a simple structure.

Work carried out to date has shown that large, planar, dilated, young, features with minimal jointing, low cohesion, and that are not cut by anything else are the most dangerous faults. As a generalisation, the thicker a dyke is, the stronger it is. Dykes that are 15-20 m wide or more are seen to be less of a problem because pillar stresses are more widely distributed. Events with magnitude greater than 4.5 are associated with large scale structures i.e. with large throws. Even if a magnitude +4.0 event occurs it is difficult to know whether an area had been completely destressed. Large magnitude events recorded in the Klerksdorp area show that these events are most likely to be driven by regional tectonic stresses. The strength of the structure and its dip were also identified as being important.

Bracket pillars should either be wide - i.e. 40m or not be left at all since under-designed pillars are potentially extremely hazardous. Numerical modelling shows a big difference between pillars that are 10 and 20 m wide but very little difference for pillars of greater width.

Practitioners also commented on the benefits of mining through geological structures first, or mining on retreat away from such structures so that difficult areas are mined out first while stresses are relatively low.

There are also the problems of not being able to model bedding planes with elastic codes nor being able to take dyke material into account with such codes. Structures with large throws have to be modelled with a grid that is too coarse to determine behaviour on a small scale.

In the course of the industry survey 51 bracket pillar case studies were collected. These were collated and classified into two groups - stable or failed pillars. This classification was based purely on short term stability, in other words, if the reef within the area could be successfully mined i.e. without a major rockburst after leaving a bracket pillar, then the pillar was classified as stable. On the other hand if a major rockburst occurred during mining of the reef in that area then the pillar was classified as 'failed'. This is obviously not an ideal classification, since it does not take long term stability into consideration, and the definition of a major rockburst is subjective, but it does provide some useful insights.

Figure 1 shows the data for cases of bracket pillars left against faults. Wherever possible a likely explanation for the failures is given. Cases where the width of the bracket pillar was changed as a result of a rockburst are denoted by means of an arrow. Figure 2 shows the data for cases of bracket pillars left against dykes.

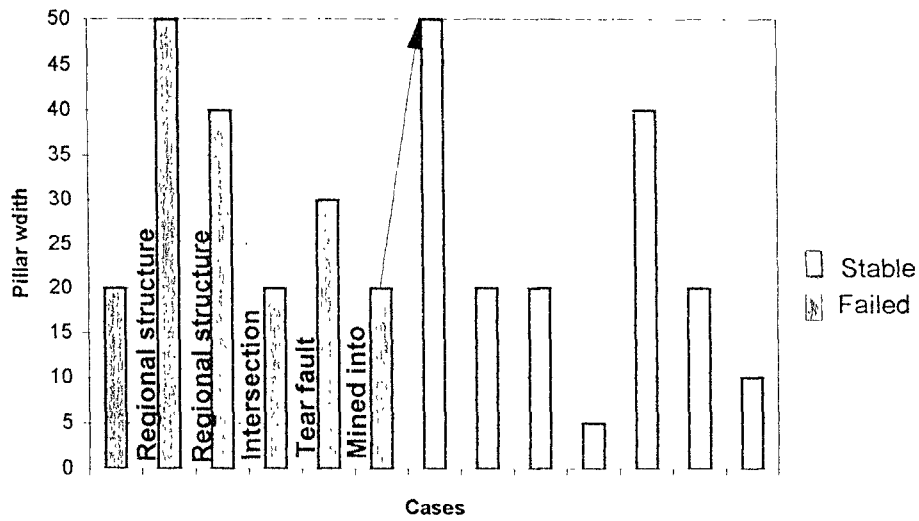


Figure 1. Failed and stable faults. Arrows indicate a change in width of a bracket pillar adjacent to a particular structure

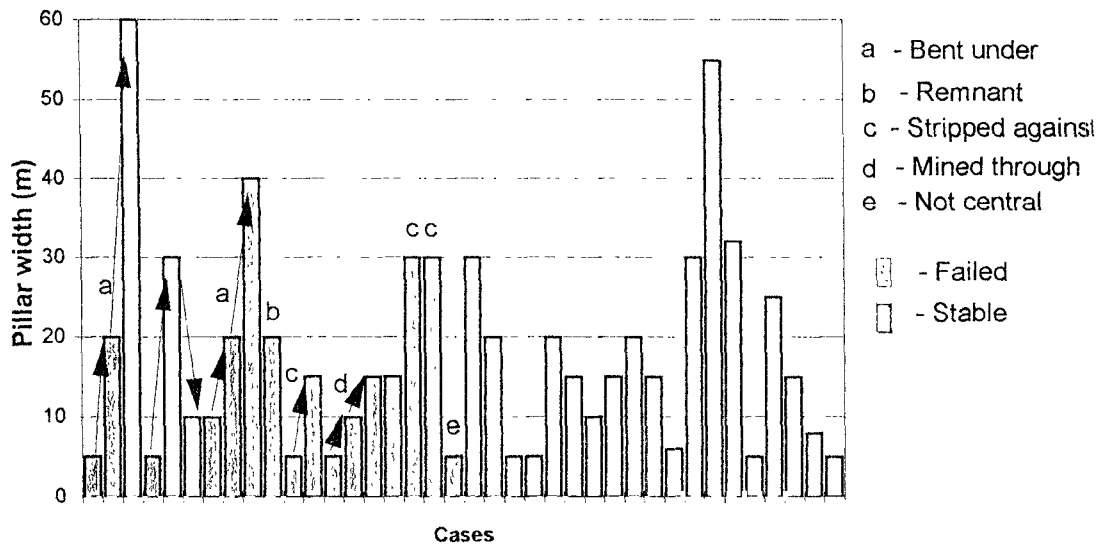


Figure 2. Failed and stable dykes. Arrows indicate a change in width of a bracket pillar adjacent to a particular structure

Analysis of these cases provides some understanding of the behaviour of bracket pillars. In some cases bracket pillars seem to be the best strategy for mining in the vicinity of a geological structure and in others, alternative rockburst control techniques may be more applicable. The following situations have been identified as problematic and in these cases improved criteria are necessary:-

- i) a structure that has been stripped against or mined into on one side,
- ii) a shallow dipping structure,

- iii) a large scale regional structure,
- iv) mining of a remnant adjacent to a geological structure,
- v) intersecting geological structures and fault-dyke combinations.

#### Geological structure classification

The intention of this section of the work was to set up a database containing information relevant to the nature of dykes and faults and determine which of these had the strongest influence on the failure potential of the structure. Ideally, different layouts would be used adjacent to different structures based on the nature of the structure. The parameters that were identified as being potentially important are grouped into two categories:-

- 1) The characteristics of the structure - in particular its age, orientation, composition, strength, fault association, throw, width, frictional strength, elastic or plastic nature.
- 2) The characteristics of the environment in which the structure exists, in particular, the mining induced stresses, the k-ratio and the properties of the surrounding rockmass.

A number of problems associated with this type of work were identified and these are listed below:-

- 1) Limited data - very little information relating to the nature of a dyke or fault is collected on a routine basis by mine geologists.
- 2) Difficulty of distinguishing between rockbursts which occur as a result of the mining strategy in the vicinity of the structure and those that occur because of the nature of the structure.
- 3) Very limited database of rockburst damage reports
- 4) Seismic events are very rarely located to within the accuracy of the width of a fault plane.
- 5) There are so many parameters to consider that it is very difficult, if not impossible, to separate their effects.

A review of literature covering this particular topic gave the following information;

For both cases of fault and dyke classification the parameters relating to the hazardous potential of the structure were divided into three types:-

- i) properties specific to the structure itself
- ii) parameters relating to the existence of the structure within a larger rockmass
- iii) parameters that are specific to the mining layout in the vicinity of the structure - these primarily relate to the interaction of the structure with the mining induced stress fields and the extent of mining along the length of the structure.

#### Dyke data

The following parameters were considered:-

Dyke strength - In general the greater the strength the more hazardous the structure, although instances were found of hazardous dykes with UCS's of 175 and 185 MPa. - in these cases, the nature of the contact or the variation in the UCS across the structure is important.

Elastic moduli - The greater the elastic moduli the greater the strain energy that can be stored for a particular strain rate and the greater the associated hazard.

Mineralogy - Siliceous rocks seem to be more hazardous than argillaceous rocks, probably due to their brittleness. Grain size is important.

Alteration/decomposition - A number of hazardous dykes were identified as being highly altered. However, insufficient information is available at this stage to draw any firm conclusions.

Width - Although wider dykes are seen to be more hazardous than narrow dykes, no clear trend was apparent from data collected.

Dyke shape or geometry - This parameter seems to be very important - particularly in terms of the proximity of mining to any irregularities in shape e.g. bulging or bending.

Orientation - This is important in terms of the interaction of the dyke with mining-induced stresses and the in-situ stress field, however, as an independent parameter it is not relevant.

Dip - Since the majority of structures considered had very similar dips it is difficult to get an unbiased influence of dip angle.

Nature of contact - This is important - particularly with reference to shearing, welding or strength changes across the contact

Age - Younger dykes appear to be more hazardous - although this may be controlled by the material type and strength.

Throw - It seems that the existence of associated faulting is important but the actual throw is not. Larger throws are generally seen to be more hazardous but a number of cases of hazardous structures with throws of only 2 m were found.

Jointing - associated with the structure and the material surrounding the structure may be important but there is insufficient information to understand this effect properly.

#### Fault data

Throw - This is the parameter most widely used in determining the hazard associated with a structure, however, analysis of collected data indicates that it is not possible to describe the structure using only this parameter. A number of structures with small throws have been found to be hazardous.

Dip - In general the shallower the dip of the structure the more hazardous it is.

Fault contact - The presence of mylonite affects the potential of and the way in which a fault will fail. The presence of and nature of fault gouge may also be important

Fault geometry - This is difficult to measure and just about impossible to make a generalised conclusion about but it is important. Irregularities in shape result in asperities which may become potential rupture points. Other aspects that need additional consideration are fault planarity, continuity and surface regularity.

Lithology on either side of the fault - Particularly hazardous structures have materials with markedly different strength characteristics on either side of the fault surface.

The mining layout will influence the hazard associated with the structure in terms of the following parameters:-

- i) presence of mining on both sides of the fault,
- ii) whether the structure is stripped against or not,
- iii) the length of the structure that is exposed,
- iv) direction of mining - i.e. towards or away from the structure,
- v) the orientation of mining induced stresses relative to the structure,
- vi) the k-ratio in the area.

Table 1 - Variation in parameters for so-called 'Hazardous' and 'Non-hazardous' dykes.

<u>Parameter</u>	<u>'Non-hazardous'</u>	<u>'Hazardous'</u>
UCS	50-300 MPa	175-414 MPa
Throw	0-20 m	2-45 m
Alteration	40-53 %	20-90 %
Width	4-40 m	5-60 m
Dip	70-90°	60-90°
Young's modulus	100	65-100
Poisson's ratio	0.31	0.22-0.31
Shear modulus	31 MPa	27-41 MPa
Density	3028	2813-3012
Orientation w.r.t. strike	19-75°	0-65°

Table 1 lists some of the parameters contained in the database. Unfortunately since so-called 'non-hazardous' structures are generally not problematic, very little information is collected about them. This is evident in the fact that there are no ranges given for a number of parameters listed in this group.

### Numerical modelling

Numerical modelling of bracket pillars was carried out in two parts - the first being back analyses of a number of layouts in order to determine the reliability of current criteria; the second being 2D modelling of an idealised vertical dyke structure in order to determine the influence that varying dyke stiffness has on the stress distribution around a hypothetical stope.

Problematic areas identified during the industry survey were back analysed. These included:- the intersection of a number of structures, a combined fault dyke structure, an inaccurately located structure, a structure previously mined through and geological structures within remnants. These results were analysed in terms of the variation in the extent of the positive ESS lobes with successive mining steps and the interaction of the maximum principal stresses within the structure.

In all cases modelled seismic moments were corrected by subtracting recorded seismic moments during the previous time period. However, even with this correction the modelled seismic moments exceeded those recorded in a number of cases.

FLAC was used to model an advancing stope approaching a dyke for two different dyke types. In the control case the dyke had the same stiffness as the surrounding rock mass, which was modelled as a hard quartzite, while in the second case the dyke material was significantly stiffer than the surrounding rockmass. Analysis of the stress profiles as mining approached the dyke showed that there was a marked stress discontinuity just beyond the dyke host rock contact in the case of the stiff dyke. This was not evident in the control case. In addition, it was found that the maximum ESS in the case of the stiffer dyke material was of the order of 10 to 15 MPa greater than for the control case (Figure 3).

For the stiffer dyke material, the stresses in the bracket pillar are lower, while the stresses in the dyke are higher by about 40 MPa along most of the width of the dyke. The dyke is thus acting as a stress concentrator(Figure 4).

### Seismic analysis

As a result of a suggestion to incorporate intensive seismic monitoring into this research project some work has been carried out on back analysing existing seismic records from areas of mining in the vicinity of geological structures. The objective of this research was to determine whether it is possible to characterise structures based on their seismic behaviour. Back analyses of such data could also increase understanding of the relationship between seismicity associated with a geological structure and the distance of current mining from that structure.

## COMPARISON OF ESS BETWEEN CONTROL CASE AND STIFF DYKE

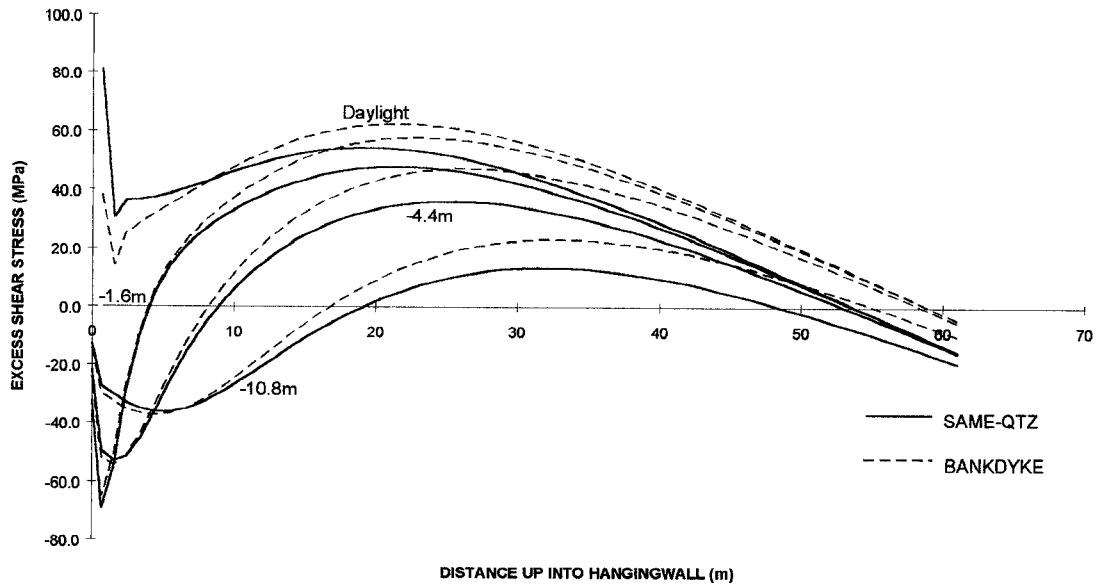


Figure 3. ESS values along the dyke-quartzite and quartzite-quartzite contacts. The value and area of ESS is higher in the case of the dyke-quartzite situation

## COMPARISON OF STRESS PROFILES BETWEEN CONTROL CASE AND STIFF DYKE

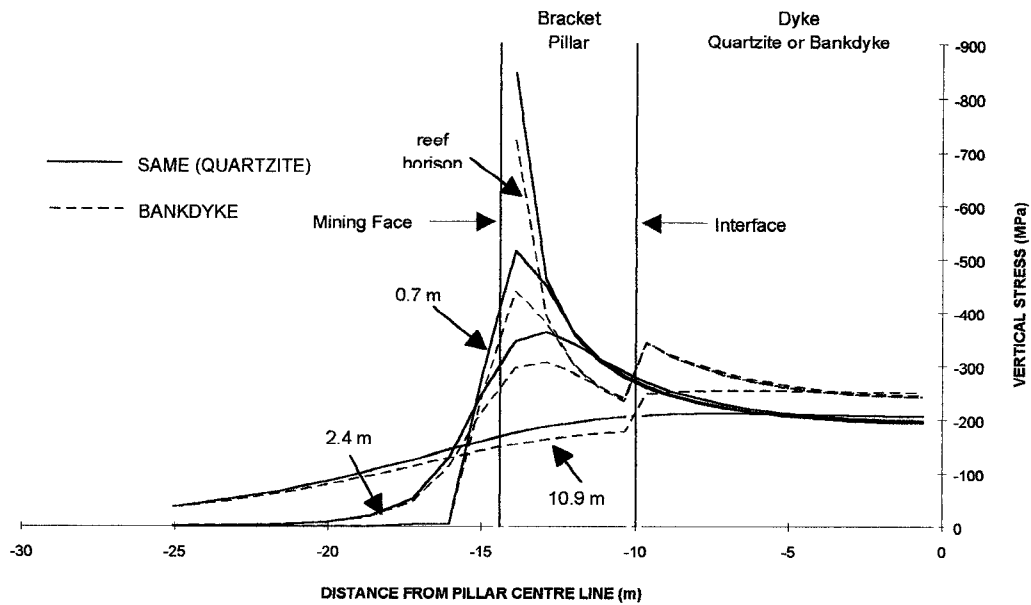


Figure 4. Vertical stress along horizontal lines in the hangingwall of a stope with quartzite ahead of it and a dyke ahead of it. There is a discontinuity at the quartzite-dyke contact and higher stresses in the dyke



The back analyses were carried out by dividing the area of interest into a number of strike or breast panels. For each panel the average distance of the face to the structure was then plotted. The distance between seismic events occurring within the bounds of each panel and the structure were then measured and these superimposed on the face distance information.

Cases considered include:-

- Intersecting structures
- A combined fault - dyke structure
- Mining through a structure.

The results from the analyses of intersecting structures are presented below.

Because of the size of the database only events with magnitudes greater than 1.0 were included in this analysis. These curves were plotted up for a number of successive time periods enabling one to see the variation in these curves with decreasing distance from the geological structure.

#### Case Study:- Intersecting structures

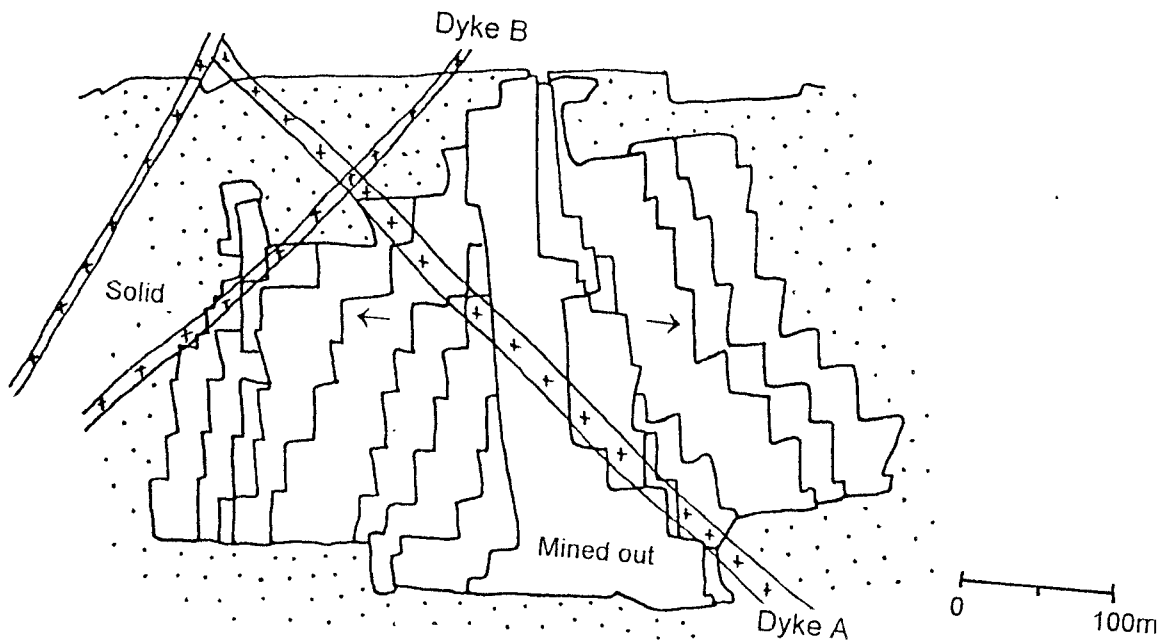


Figure 5- Mining layout in the vicinity of the intersecting structures

This study was primarily concerned with the relationship between the active mining faces and Dyke A and Dyke B. Several of the dip panels mined through Dyke A very early on. The area was divided into 11 breast panels, and seismicity and face positions at three monthly intervals were used in the analysis. The resulting graphs are shown in Figures 6a to i.

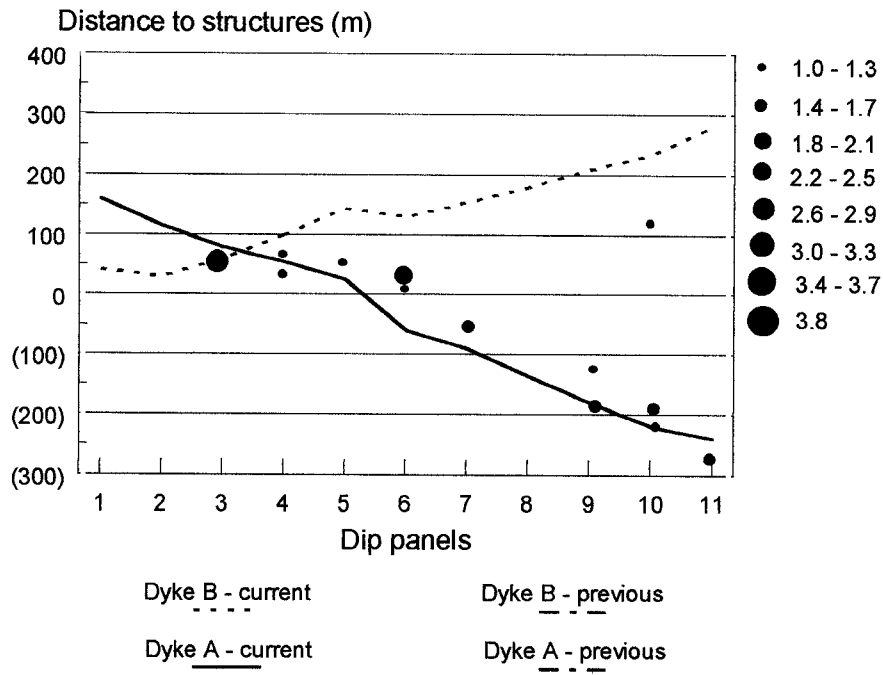


Figure 6a - April to June 1992

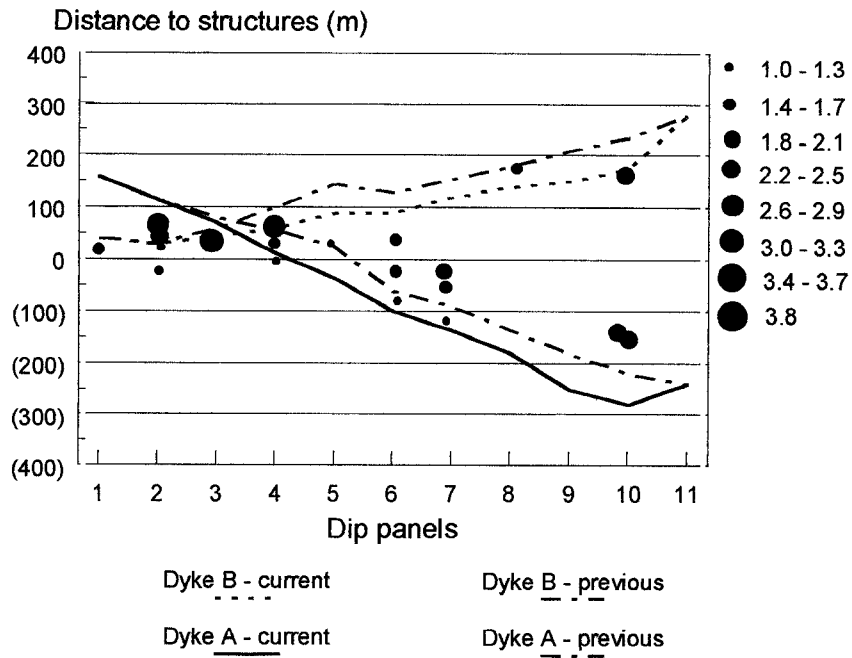


Figure 6b- July to September 1992

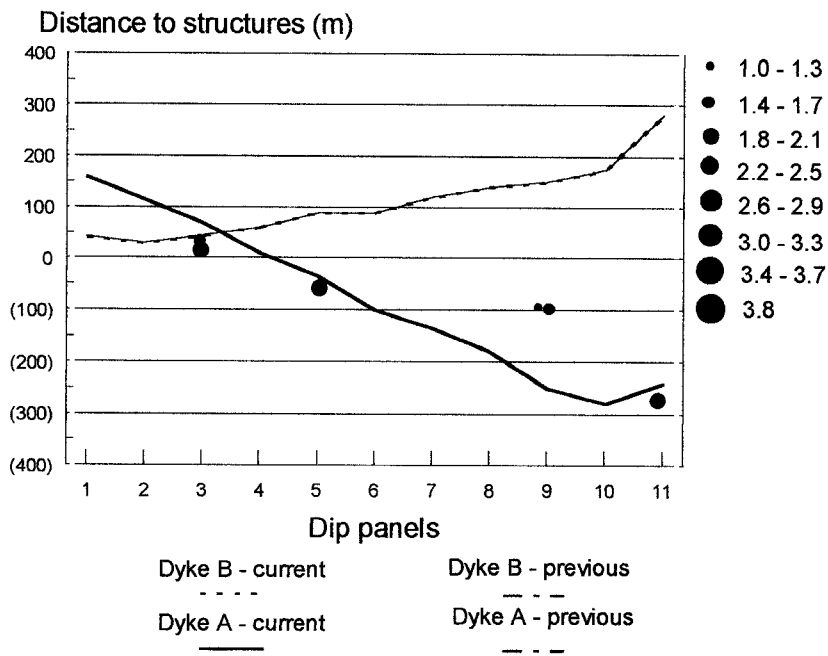


Figure 6c - October to February 1993

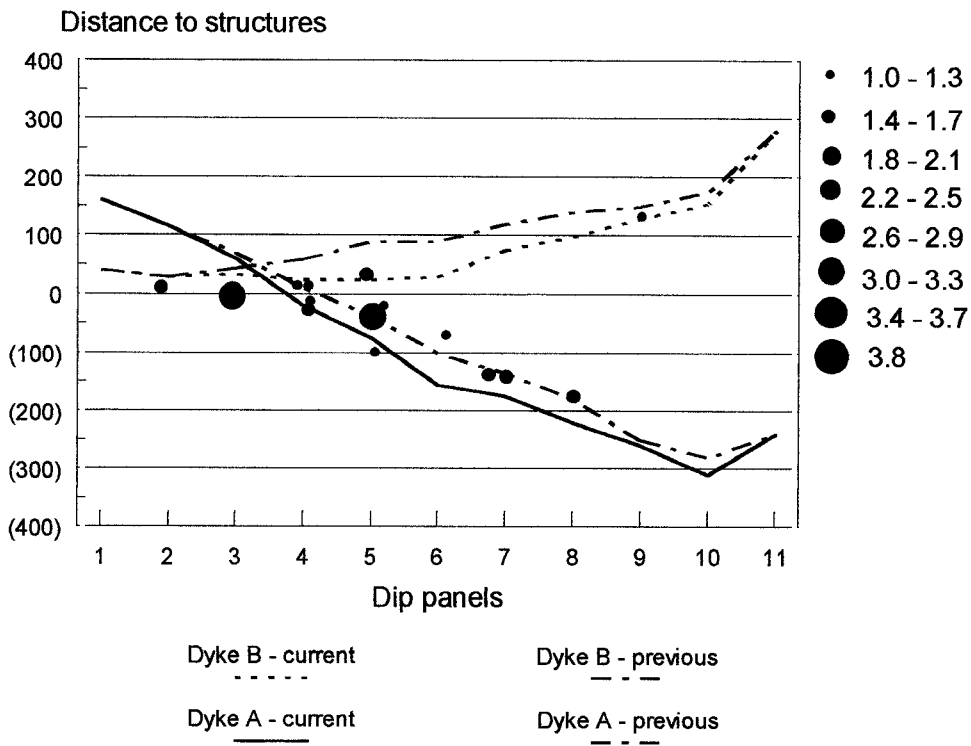


Figure 6d - January to March 1993

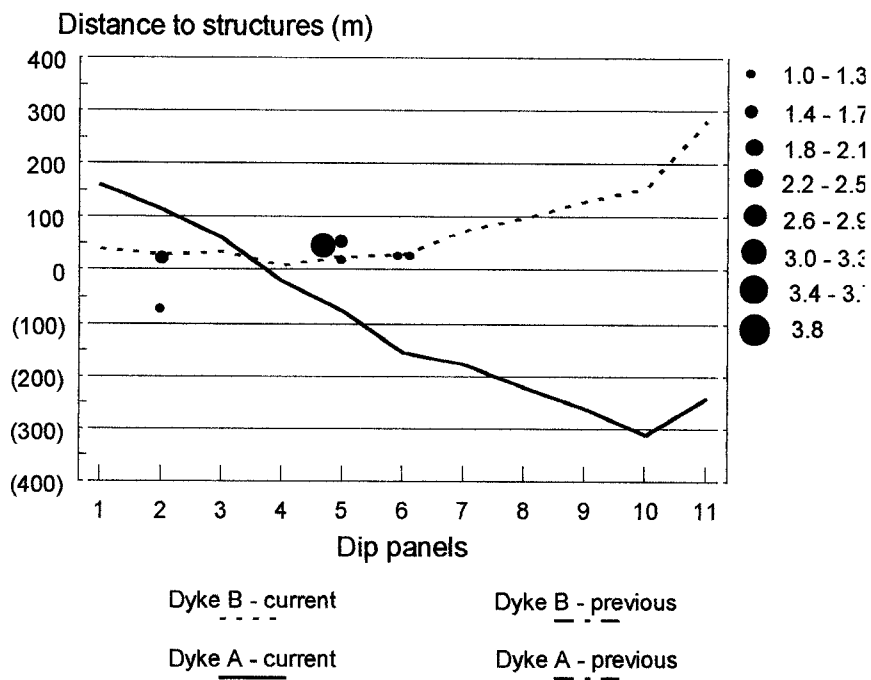


Figure 6e - April to June 1993

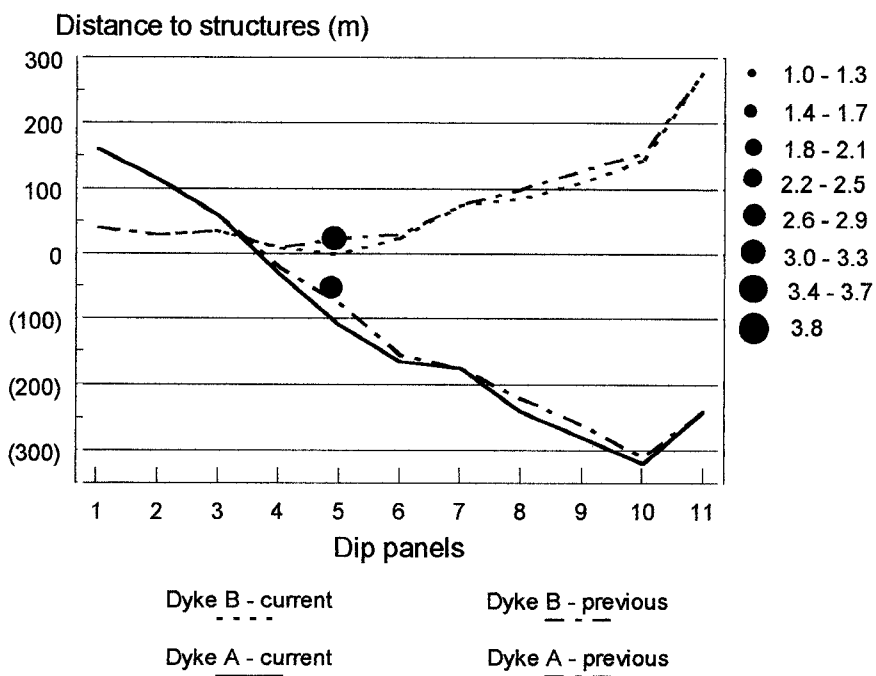


Figure 6f - July to September 1993

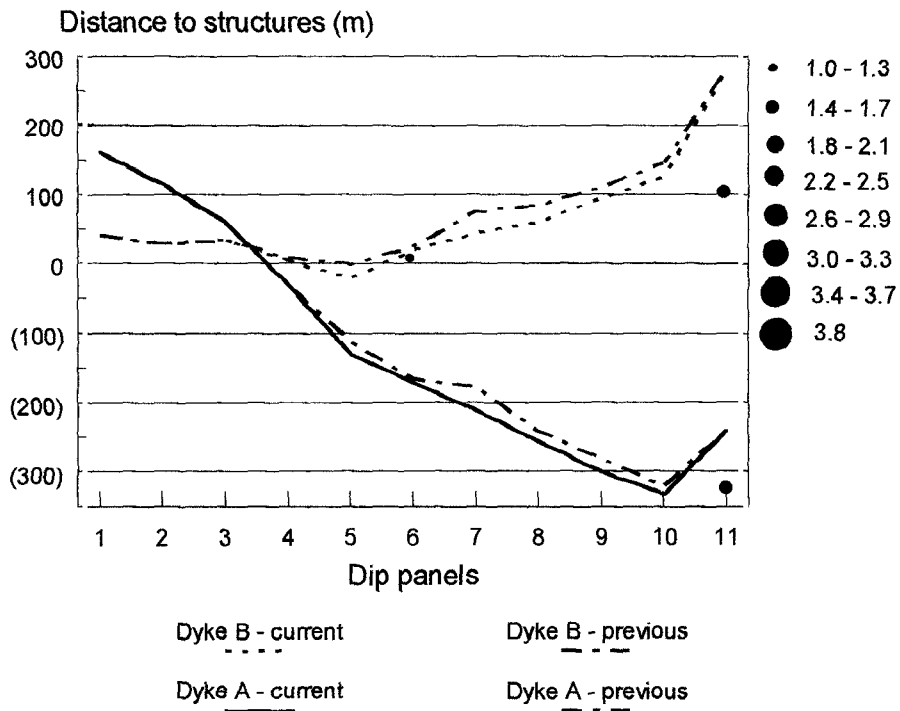


Figure 6g - October to December 1993

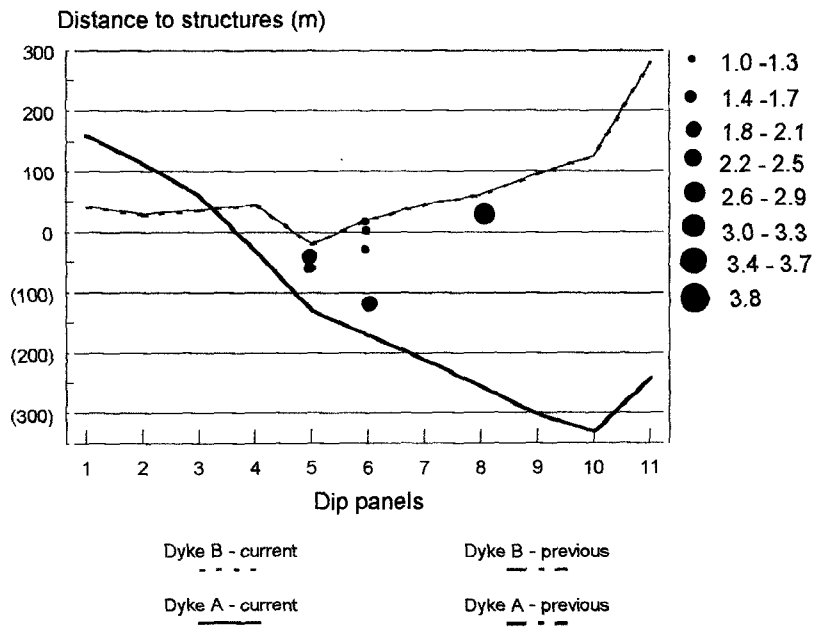


Figure 6h - January to March 1994

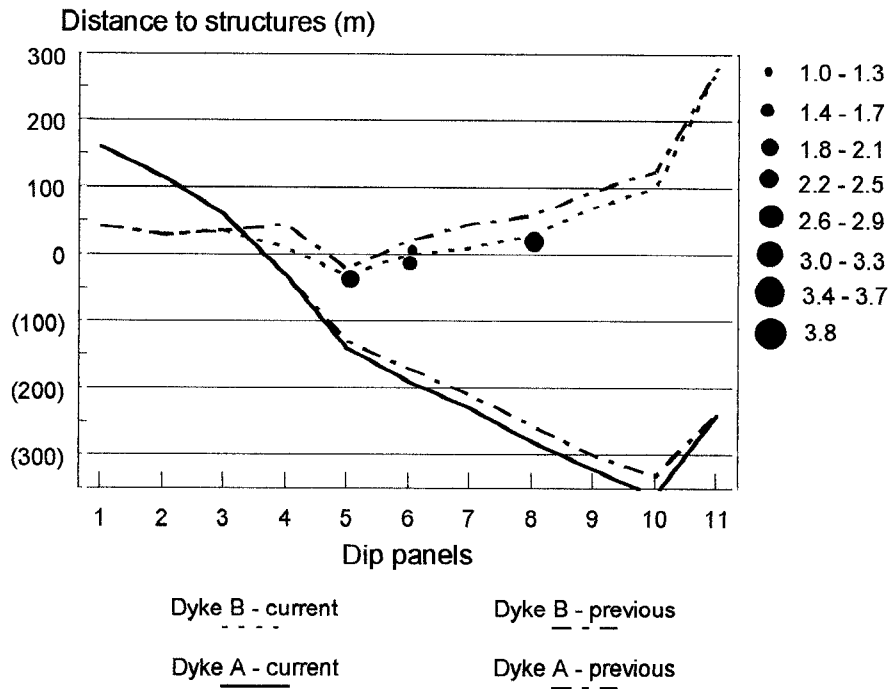


Figure 6i - April to June 1994

Figures 6a to i - Panel to dyke distances with event to dyke distances superimposed.

Based on the seismic records associated with mining it appears that the largest magnitude events occurring in this area are associated with mining faces that approach within 20 to 30 m of the dyke structures. The largest magnitude events do not appear to distress the area being considered and a number of events with magnitudes greater than 3.0 occurred during successive time periods. During the last two time periods considered it appeared that the focus of activity was migrating further down dip as deeper panels approached within 30 m of Dyke B. Since the seismicity associated with this structure does not decrease once mining gets closer than 20 m from the structure, a bracket pillar with a width of at least 30 m should be left. Based on this data it is apparent that using the current mining layout in the vicinity of these structures is very hazardous and even if it is not possible to use a bracket pillar adjacent to this structure in other areas, it is imperative that the mining layout be modified.

Similar studies were conducted for mining through a structure and mining in the vicinity of a combined fault - dyke structure. Based on the analysis of the former data it appears that the seismicity associated with this structure increases dramatically when mining approaches to within 20 m of the structure. The largest event occurred after a number of panels had mined through the structure. This event was sufficiently large to distress the dyke in that very little seismic energy was released during the next 12 months even though there was a significant amount of mining in the area during that period.

From an analysis of the data from the dyke fault case it is apparent that any increase in seismicity associated with this structure should only occur when mining is less than 30 m from the structure. Unfortunately seismic data is not available for the time period during which the panels closest to the structure were mined. During the remainder of the mining considered the maximum events recorded were around 2.2 and in general the seismicity pattern is constant.

Further detailed information on these case studies are reported in a SIMRAC report (D A Selfe, 1995)

### Discussion

While carrying out this research it became obvious that an essential part of a mining strategy for mining in the vicinity of geological structures would have to address the problem of mining through structures, since on any mine there will always be structures that cannot be bracketed for whatever reason.

The 30° rule should be used whenever possible and should not simply be applied to individual panels but rather to the orientation of the entire longwall. In addition face shapes should be kept as regular as possible. It may also be necessary to change the face shape prior to mining through a structure - in one case the faces were pushed into an arrowhead shape in order to minimise the positive ESS lobes during mining of the structure. This layout ensured that there was a limited face length cutting through the dyke initially. This preliminary cut through the dyke allowed some stress associated with the dyke to be released prior to the period when extensive face length was within the dyke

Analysis of past seismicity patterns associated with the particular structure can provide invaluable insight into the likely behaviour of the structure during current mining.

In addition the support density should be increased while mining is in the immediate vicinity of the structure. In some cases stiffer support should also be used in order to reduce inelastic rock movement. Wherever possible features should be mined away from and not breasted onto. Spans should be kept as small as possible.

### Conclusions

Based on the information gathered to date it is apparent that it is not yet possible to quantify the hazardous potential of a geological structure based on its characteristics. Unfortunately it will take many years to produce a database which would make such a quantification possible. It is obvious that neither all faults nor all dykes behave in the same way. However, it is not possible to incorporate this variable behaviour into the numerical modelling programs that are currently used to design mine layouts. It is therefore almost impossible to design bracket pillars reliably around different structures using only numerical modelling. Back analyses of a number of cases shows that the current criteria are still useful tools in designing bracket pillars. In the majority of cases considered, the modelling over-predicted the hazard associated with a structure for 70 % of time periods considered. In order to increase this reliability it is necessary to incorporate additional information into the design process.

Analysis of seismic data is a powerful tool that can be used in this regard. Using this data it is possible to quantify the variable behaviour of different structures. By obtaining a graph for each time period it may be possible to determine whether large events associated with a particular structure serve to destress that area or not and the extent of redistribution of stress after such an event. Using this information it may be possible to predict the behaviour of the same structure in a different area or in the future. It is also possible to determine the distance at which seismicity associated with that structure increases. This information should be used in conjunction with numerical modelling to design mining layouts particularly for determining bracket pillar sizes. Based on the seismic history of a structure it should be possible to classify the structure into a particular type and design the mining layout based on rules applicable to the type.

Analysis of stable and failed bracket pillar cases has shown that there are a number of situations which are likely to be very hazardous. Whenever such situations are identified additional

precautions should be taken. In all cases where mining is in close proximity to a geological structure special attention should be paid to factors such as face shapes, leads and lags and mining configuration.

## STABILISING PILLARS

### Literature survey

In order to appreciate the current approach and associated problems with the use of stabilising pillars in the South African mining industry it is important to consider the philosophy behind the development of these structures and therefore a literature and industry survey were conducted.

The industry survey revealed that five Witwatersrand gold mines employ strike stabilising pillars for regional support in their underground layout. Published reports on the design and implementation of such pillars are available for four of these mines, namely E.R.P.M., Blyvooruitzicht G. M., Kloof G.M. and Western Deep-levels Limited (W.D.L.).

#### East Rand Proprietary Mines Limited

The first strike stabilising pillars were introduced at E.R.P.M. between 1963 and 1966. At the time the depth of mining was approaching 2400 m below surface. These pillars were 70 to 80 m wide with an accompanying stope span in the region of 270 m. In the late 70's this design was reviewed and their width reduced to between 50 and 60 m at spans of 250 to 300 m. This design afforded a high extraction ratio while limiting the average pillar stress average pillar stress (A.P.S.) to 400 MPa.

#### Blyvooruitzicht Gold Mining Company Limited

The implementation of strike stabilising pillars at Blyvooruitzicht commenced in 1977 when management noted their benefits to safety and production as experienced on E.R.P.M.. With energy release rate (E.R.R.) as the significant design factor, a 30 m wide, stable boundary pillar, separating the mine from W.D.L., was back analysed revealing an associated A.P.S. of 600 MPa. It was deemed necessary to include a safety factor of 1.5 in the design of their stabilising pillars; limiting the A.P.S. to a maximum of 400 MPa. For a stoping width of 1.2 m this criterion gave rise to 45 m wide pillars that were approximately 225 m apart with an accompanying width to height ratio of 36:1. The design included 30 m dip ribs which were to form a final separation between advancing faces. For economic reasons, and from a safety point of view, this design took into consideration the geological features within the area, bracketing potentially unstable geologic features. That is, the dip ribs are essentially bracket pillars. It was estimated that this layout reduced the total E.R.R. by 44%, to give an average calculated E.R.R. of 30 MJ/m<sup>2</sup>, and that the design should be valid to depths of 3000 m, beyond which the width should be increased. This design was derived assuming that the rock mass behaves elastically.

#### Kloof Gold Mining Co. Limited

The area above the 2000 m elevation at Kloof was mined on a scattered mining basis. When the mined-out spans in this region became extensive a serious rockburst problem was experienced. Consequently, in the mid seventies, when longwall mining commenced below the 2000 m level stabilising pillars were included in the mine layout. Design of these pillars took into account A.P.S., the expected reduction in closure, the spacing between mining levels and the number of levels per mini-longwall. The A.P.S. design criterion was, and still is, to limit A.P.S. to 2.5 times the Uniaxial compressive strength (U.C.S.) of the host rock, where the A.P.S. is estimated using elastic analysis. The resulting pillars were 60 m wide in the plane of the reef with a centre-to-centre spacing of 360 m. Computer simulations performed at the time estimated that the E.R.R.



was reduced by about 30%. The design of subsequent stabilising pillars included a width to height ratio criterion ranging from 30:1 to 40:1, where the height considered is based on the concept of increased effective height of the pillar due to ancillary gully excavations and mobile partings.

Experience gained from mining with stabilising pillars, and through the introduction of a seismic network, has shown that the pillars on Kloof have remained stable and have reduced the occurrence of large magnitude seismic events. It was reported that a smaller number of large magnitude events are recorded at Kloof in comparison with other gold mines in the district.

#### Western Deep-levels Limited

Cutting of 20 m wide pillars on the Carbon Leader Reef (C.L.R.) commenced early in 1980. Initially the pillars had 8 m wide ventilation slots cut through them at 120 m intervals. The 8 m slots soon proved to be too narrow and were increased to 16 m. With the introduction of stabilising pillars the layout of travelling ways and other developments had to be reconsidered and a more practical layout established. Within the first six years of their implementation 26 new pillars were cut on the C.L.R.. It was found that the decrease in reserves as a result of the introduction of pillars did not have a negative effect on the productivity of the mine.

Although the rockburst incidence was reduced there were a number of serious problems associated with these narrow pillars.

- i) The panels above the pillars suffered severe and continuous damage while those below the pillars suffered very little damage. On the up-dip side of the pillars a hangingwall stability problem had developed causing an increase in the number of rockfalls.
- ii) From within the ventilation slots it could be seen that the pillars were fractured throughout leaving no solid core and that appreciable deformation was taking place.
- iii) Cutting of the ventilation slots proved very difficult and there were often rockbursts when doing so.
- iv) It was difficult to maintain pillar widths and pillars often ended up being less than 20 m wide.
- v) Some of the pillars were failing, the seismicity associated with failure showed dilational first motions indicating that they were possibly failing as a result of crushing.

As a result of these problems, in mid 1985 the pillar width was increased to 40 m with a 280 m centre-to-centre distance. This design still enabled an 85% extraction ratio.

W.D.L. is the only mine to have reported problems due to pillar instability. Pillar failures are the second largest cause of seismic events with magnitudes in excess of 3.0 on W.D.L. and their frequency increases with depth. Such events rarely affect the mining face area since they tend to locate some distance behind the face; although there are still occasional events which do locate close to the face. These large events are however, still a safety hazard as they can cause serious damage to footwall haulages and other service excavations in the vicinity of the pillar. Observations and investigations at pillar foundation failure sites have revealed a wealth of information regarding failure mechanisms, although the analysis of seismic data has proved to be the most valuable tool. Many pillars with a history of failure appear to fail at regular intervals as mining progresses, indicative of a step-wise release of energy. Hence there appears to be a 'critical' face advance associated with pillar failure events. At W.D.L. large pillar foundation failure events tend to occur after every 60 to 100 m of face advance. Since pillars inhibit stope closure the result is an increase in shear-stresses along the edges of the pillar. After a certain critical face advance, stresses in the footwall and hangingwall exceed the strength of the fractured rockmass causing sudden closure of the stope. With each major pillar event the E.S.S. maximum is shifted to a new location closer to the advancing stope face. As can be expected, modelling results show that the E.S.S. on stabilising pillar edges increases with depth implying

that the foundation failure problem is likely to worsen with depth and that failures will occur closer to the face.

### General Comments on Stabilising Pillars

A.P.S. and E.R.R. are inadequate design criteria if considered on their own. *In situ* stress-strain measurements are needed in order to obtain an improved understanding of the inelastic behaviour of the pillars and the load they are supporting, to enable more realistic numerical modelling.

Stabilising pillars offer more flexibility to the overall longwall shape, since mini-longwalls can lead or lag the neighbouring panels without creating a problem.

### Industry survey

Rock mechanics engineers on mines using stabilising pillars were consulted with regard to the use of these pillars as a form of regional support on their mines:

Various parameters pertaining to the five mines are given in Tables 2 and 3. W.D.L. is the only mine to have experienced a major pillar stability problem. A limited number of pillar failures has been recorded on E.R.P.M. in the past.

To date the design of strike stabilising pillars has primarily been determined through the use of numerical elastic modelling techniques, including the application of maximum limits of E.R.R. on the face and A.P.S.. Past experience has also been an important aspect of their design.

Table 2. Footwall, hangingwall and reef rock types and their properties for the mines of interest.

MINE		ROCK TYPE	U.C.S. (MPa)	Young's Modulus (GPa)	Poisson's Ratio
Blyvooruitzicht	Reef	Carbon Leader Reef	210		
	Footwall	Quartzites	202	74	0.19
	Hangingwall	Quartzites	250	87	0.17
Doornfontein	Reef	Carbon Leader Reef	210		
	Footwall	Quartzites	202	74	0.19
	Hangingwall	Quartzites	250	87	0.17
E.R.P.M.	Reef	Composite Reef	285		
	Footwall	Quartzites	212	75	0.20
	Hangingwall	Quartzites	352	86	0.14
Kloof	Reef	Ventersdorp Contact Reef (VCR)	200		
	Footwall	Elsburg Quartzite	180	71	0.21
	Hangingwall	Ventersdorp Lavas - Westernaria Lava - Alberton Lava	150 340	80	0.3
Western Deep Levels	Reef	Carbon Leader Reef	210		
	Footwall	Maraisburg Quartzite	202	74	0.19
	Hangingwall	Green Bar Quartzite	250	87	0.17
Western Deep Levels	Reef	Ventersdorp Contact Reef	200		
	Footwall	Elsburg Quartzite	150 - 175	70	0.21
	Hangingwall	Alberton Lava	315	117	

From this survey it would appear that the stability of strike stabilising pillars is governed by the strengths of the footwall and hangingwall rock masses, pillar width or the ratio of pillar width to pillar height, depth of mining, and stope spans. It is possible that face advance rate could also be an influencing factor in pillar stability. W.D.L. operates with the highest face advance rate of the five mines considered.

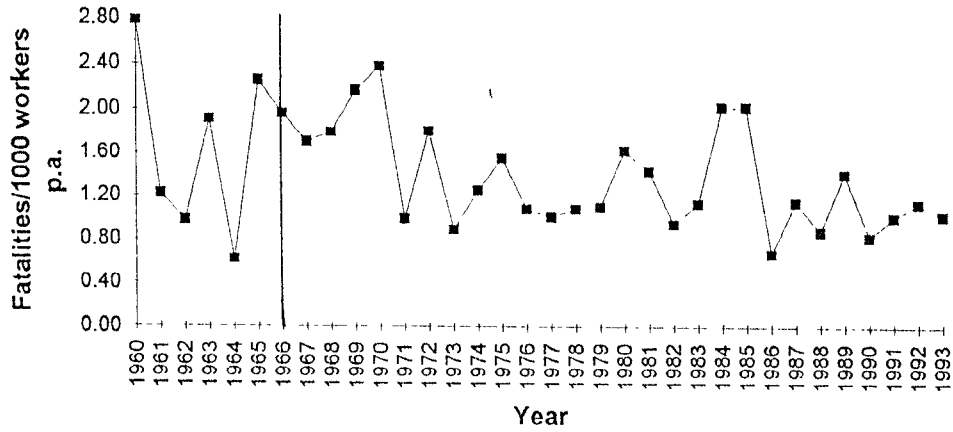
It has already been stated that stabilising pillars have improved mining conditions. In order to verify the successfulness of stabilising pillars the trends in fatality and injury rates on three mines have been examined. The mines considered are E.R.P.M., Blyvooruitzicht and W.D.L..

Table 3. Stabilising pillar dimensions and mining parameters for the mines of interest.

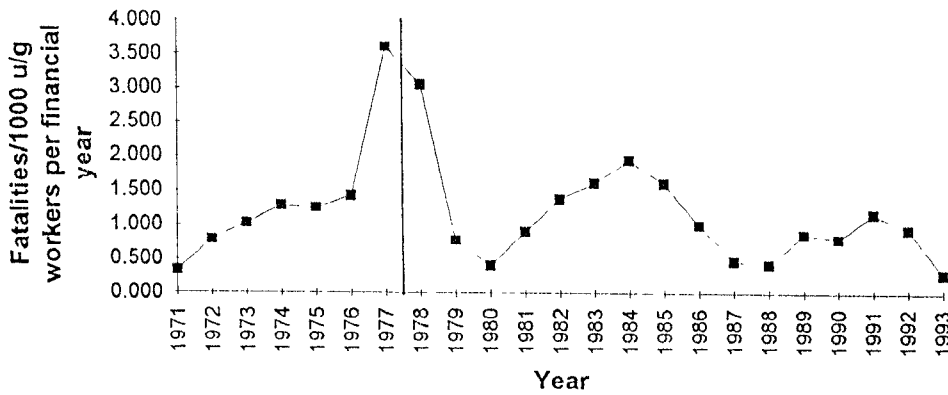
MINE	Avg. Pillar Width (m)	Avg. Stope Span (m)	Avg. Stopping Width (m)	Extraction Ratio (%)	Stope Advance Rate (m/month)	Closure Rate (mm/m advance)	Approx. Depth Range of Pillars (m below datum)
Blyvooruitzicht	45	200	0.9 - 1.1	83	4.5	36	1920 - 2620
Doornfontein	40		1.15	85	4.5 - 5	1.3 - 0.34 (cm/day)	
E. R. P. M.	50 - 60	240	1.0	80	8	36	3060 - 3710
Kloof	60	220	1.5	80	6	15	2420 - 3050
Western Deep Levels	40	240	1.0 for CLR 1.4 for VCR	78 - 86	9.5 - 12	30 for CLR 10 for VCR	2490 - 3600 for CLR 2280 - 2600 for VCR

The total number of fatalities and reportable injuries, per annum, as a result of pressure bursts and falls of ground on the different mines were obtained from the Chamber of Mines. Prior to 1980 accidents resulting from both pressure bursts and falls of ground were totalled and recorded as accidents due to falls of ground. To maintain consistency, accidents as a result of both pressure bursts and falls of ground since 1980 have been summed. At this point it is worth noting that according to the *COMRO Industry Guide* there is little, or no, correlation between fall of ground hazards and E.R.R. level. The following production and labour statistics were used to normalise the number of fatalities and injuries: average total labour, average underground labour, centares mined, tons of ore and waste broken underground, and tons milled. The annual production and labour statistics were obtained directly from the mines concerned. In the case of E.R.P.M. and W.D.L. the above mentioned statistics are on a calendar year basis. However, for Blyvooruitzicht these statistics were quoted for their financial year, which runs from 1 July to 30 June. The results are displayed in Figures 7 to 10 where the vertical solid lines mark the introduction of stabilising pillars on the various mines. Figures 8, 9 and 10 indicate that, with the exception of E.R.P.M., the normalising variable used does not effect the general trend in accident rate.

Fatalities per 1000 workers per annum at ERPM



Fatalities per 1000 underground workers per financial year at Blyvooruitzicht



Fatalities per 1000 underground workers per annum on WDL North

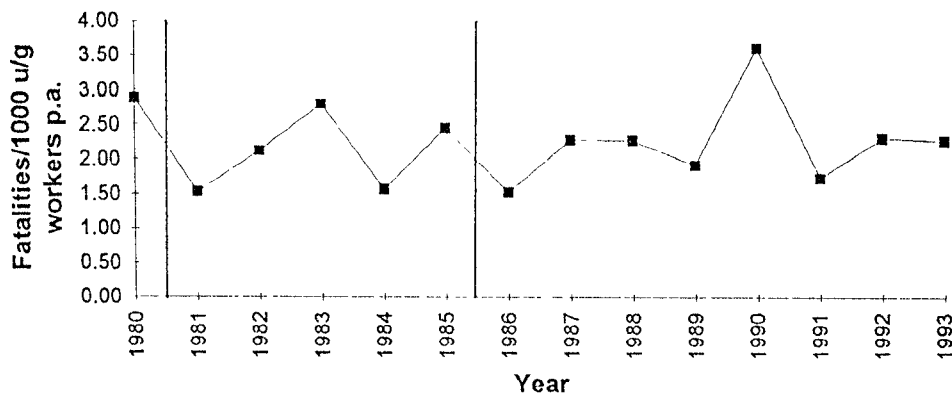


Figure 7. Fatality rates per 1 000 workers (or underground workers) on E.R.P.M., Blyvooruitzicht and W.D.L. mines. The solid vertical lines mark the introduction of stabilising pillars.

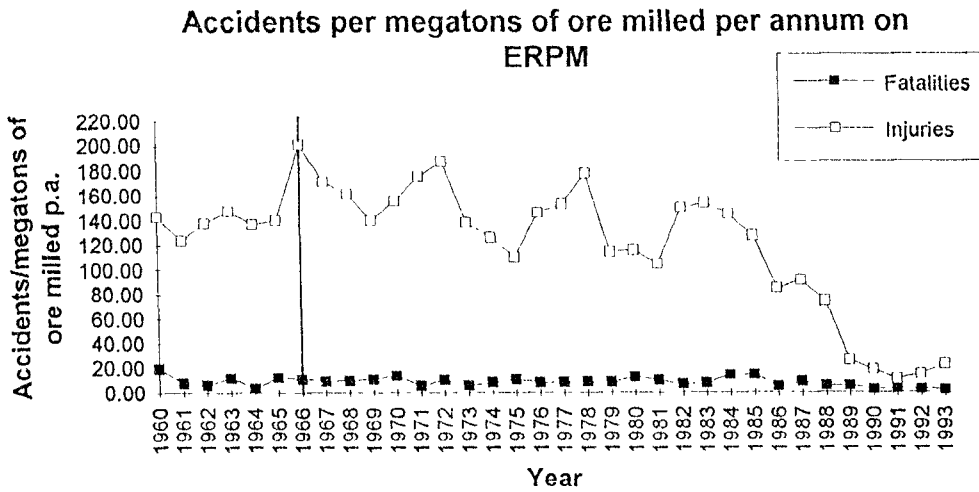
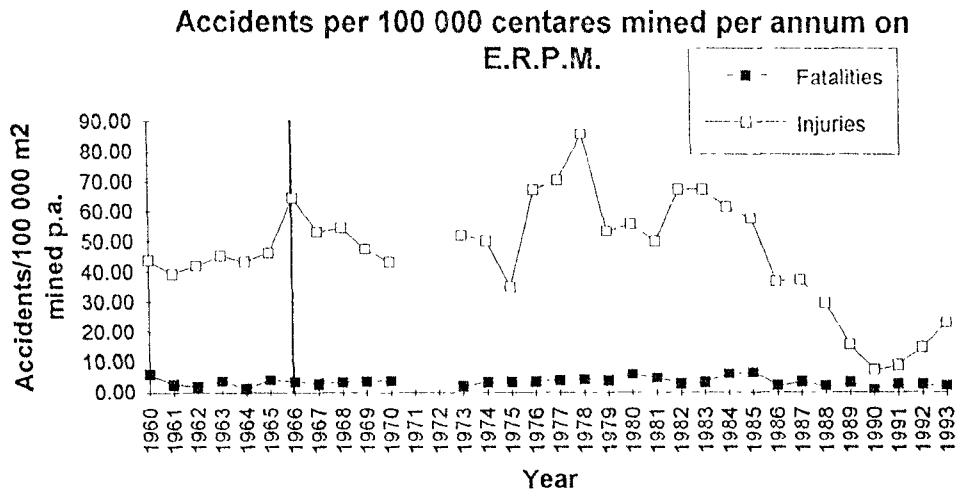
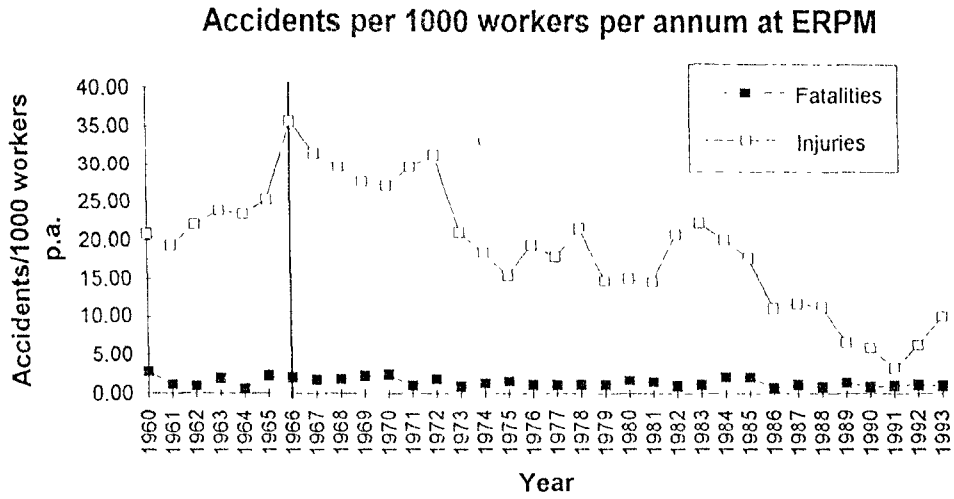
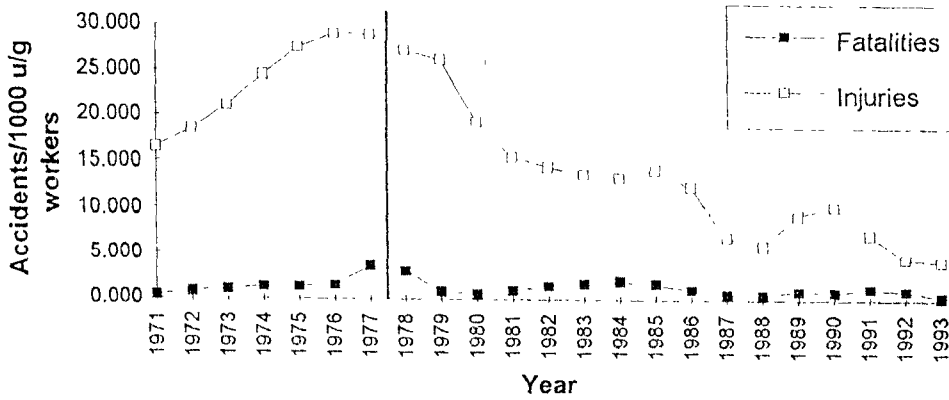
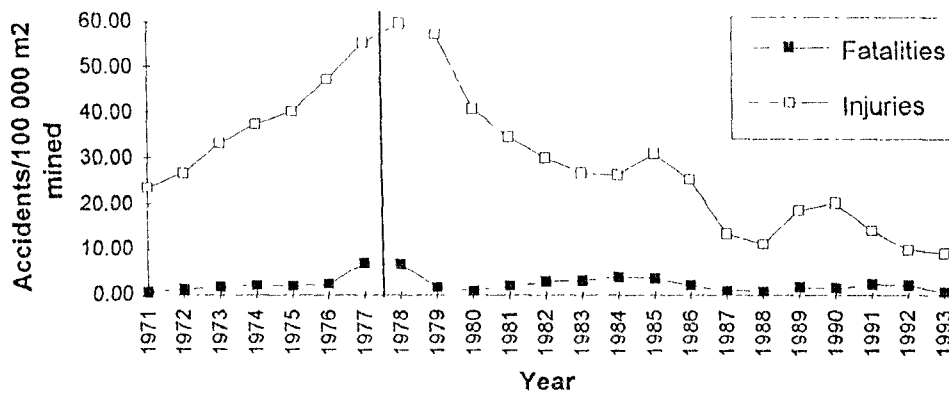


Figure 8. Accident rates on E.R.P.M. using different normalising variables. The solid vertical lines mark the introduction of stabilising pillars.

Accidents per 1000 underground workers per financial year on Blyvooruitzicht



Accidents per 100 000 centares mined per financial year on Blyvooruitzicht



Accidents per megatons of ore and waste broken underground per financial year on Blyvooruitzicht

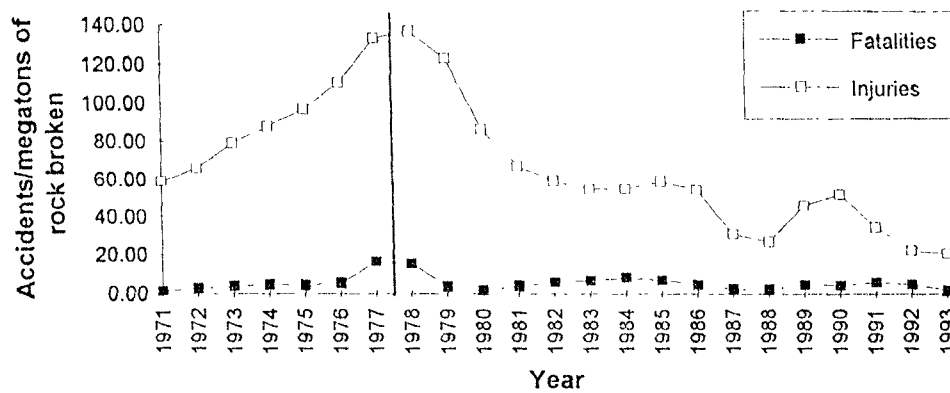


Figure 9. Accident rates on Blyvooruitzicht using different normalising variables. The solid vertical lines mark the introduction of stabilising pillars.

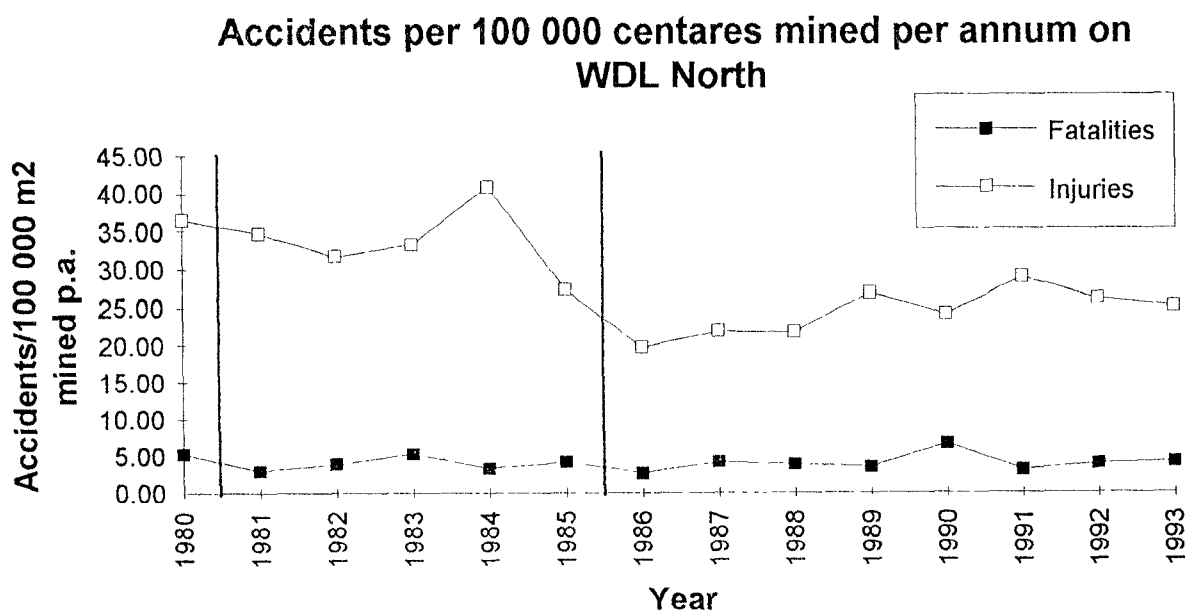
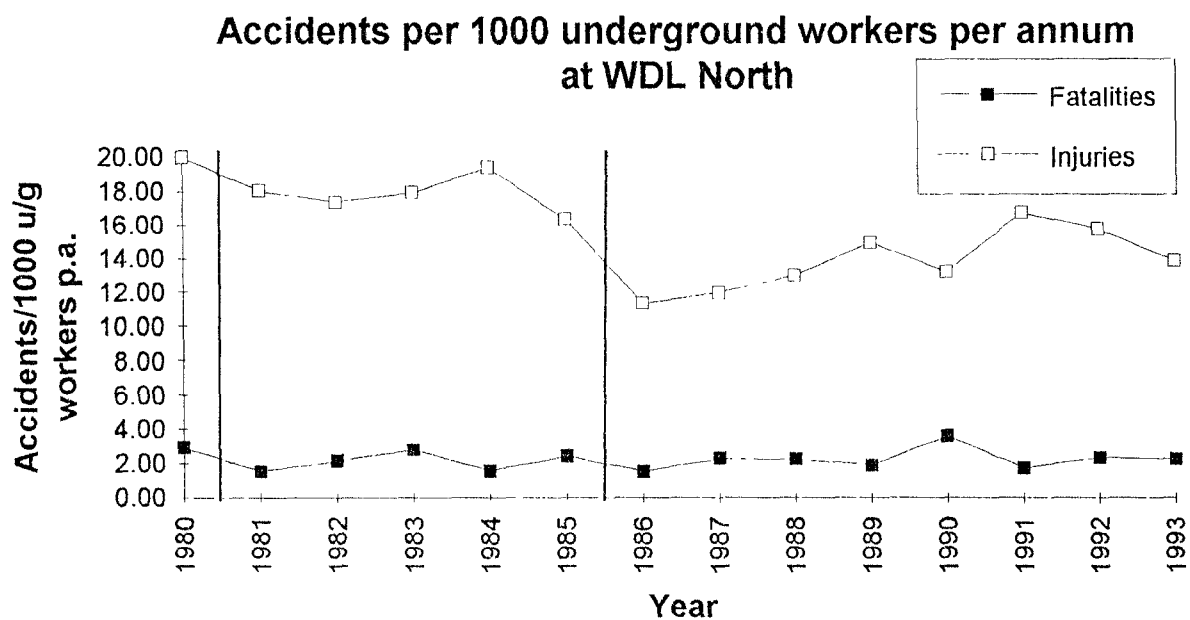


Figure 10. Accident rates on W.D.L. North using different normalising variables. The solid vertical lines mark the introduction of both 20 and 40 m wide stabilising pillars on the mine.

From Figure 7 it can be seen that, in general, on all three mines there is no notable reduction in the fatality rate following the implementation of stabilising pillars. However, from Figures 8 and 9 it is clearly evident that the introduction of stabilising pillars coincides with the onset of a general decreasing trend in injury rate. On W.D.L. North a similar trend is less obvious (Figure 10). These results indicate that the employment of stabilising pillars for regional support coincides with a reduction in the number of injuries as a result of rockbursts and rockfalls.

### Conclusion

Since their introduction in 1966, strike stabilising pillars have succeeded in improving underground working conditions on the deep South African gold mines where the longwall mining method is used. Two factors considered to govern pillar stability are the ratio of pillar width to stope width and the mechanical properties of the footwall and hangingwall strata. The former is of primary importance as it affects the E.R.R.; a lower E.R.R. is associated with reduced seismicity. From the experiences at W.D.L. it transpires that considering A.P.S. as a design criterion on its own is inadequate. The validity of E.R.R. as a design parameter has also been questioned. By all accounts it would appear that extensive investigations have resulted in a reasonable insight into the pillar failure mechanism, although an improved understanding of the inelastic behaviour of the rock masses involved is necessary to further this understanding. It is also believed that seismic analysis must be used to gain further insight into stabilising pillar behaviour. In view of this the seismic activity of a number of stabilising pillars on different mines is currently being monitored.

### Field Monitoring

Two stabilising pillar sites are being monitored on a systematic basis. One of the sites is at Kloof mine, approximately 2900 m below surface. The other site is in Western Deep Levels South mine approximately 2210m below surface. Ventersdorp Contact Reef (VCR) is being mined at both sites. A monitoring strategy for both pillars has been designed to obtain an understanding of the specific behaviour of regional stabilising pillars at depth. The field program which is in place has involved the use of various techniques and processes, and includes the following:

Continuously dedicated monitoring components

Seismic data collection using a PSS local network around the pillars.  
On-reef *dynamic-closure*, using potentiometers connected to the PSS.

Complementary techniques:

On reef closure-ride stations in areas adjacent to the pillars.

Ground penetrating radar (GPR)

Fracture mapping of off-reef excavations below pillars

Seismic refraction

Wire extensometers

Borehole camera observations

*In-situ* stress measurements

Precise levelling of off-reef tunnels below pillars.

Closure ride measurements and analysis

Actual response of the rock mass to the mining of longwall stopes can be directly assessed in terms of the amount and type of hangingwall- footwall relative displacements. Closure-ride measurements are therefore being carried out on the reef planes at both Kloof and WDL stabilising pillar sites.



At the WDL site (Figure 11) the first measurement stations were installed in September 1993 and at least five readings per month from each station have since been recorded to date. Kloof stations were installed in January 93 with readings being recorded less regularly, due to small advance rates.

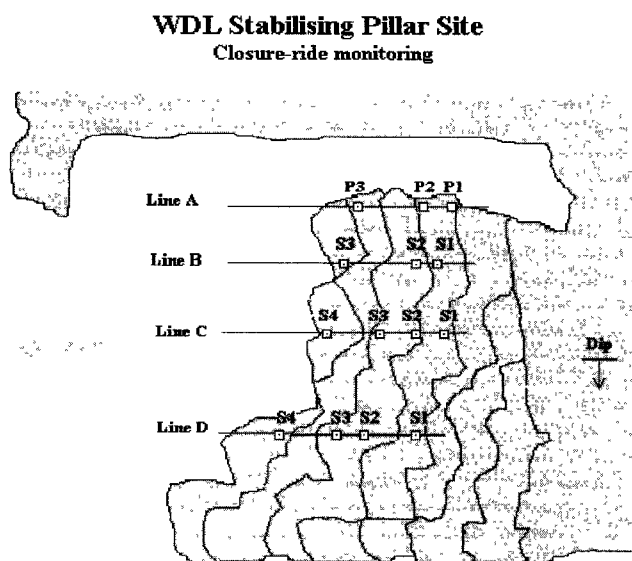


Figure 11. Relative positions of closure-ride stations and closure pots at Western Deep Levels South Mine.

Convergence pegs (S1, S2, S3, etc.) are installed as soon as the back area of a stope face is swept and no additional mining operations need to take place around the measurement positions. A significant amount of closure occurs in the stope before the stations are installed and this needs to be estimated. MINSIM-D modelling always underestimates the rate of closure in the back areas due to the assumption that the Young's Modulus of the rock mass is 70 GPa.

#### Stress measurements

*In-situ* triaxial stress measurements, using the doorstopper technique, were conducted at Kloof Gold mine to estimate the prevailing three dimensional stress state, under a mined out area adjacent to a stabilising pillar. These measurements indicated that regeneration of stresses does occur, possibly as a result of closure. A further set of measurements were carried out below the mining faces. These gave an indication of abutment stresses. Both measurements have been used to calibrate the numerical models and thus gain an idea of the stresses being carried by the stabilising pillars.

The stress measurements were carried out in the footwall, 17 metres below a mined out VCR reef. The average reef width is 1.0 m with stoping occurring at widths of between 1.2 m to 1.5 m at an average dip of 30 degrees. The reef above the selected site was mined out in May 1986. The measurements were carried out in September 1994.

#### Determination of rockmass material properties

In order to obtain rockmass material properties necessary for use in test result reduction, three samples of footwall quartzite were tested from each of the test site areas. These samples were tested using strain gauge techniques.

Test results are summarised in the table below (Table 4). On the basis of these tests an average Young's modulus of 81.55 GPa and a average Poisson's ratio of 0.19 has been used in data reduction.

Table 4. Summary of rock mass material properties used in the stress determination

Reference	Site B			Site A		
	3a	3b	3c	1a	1b	1c
Load at failure (kN)	68	75	87	99	55	75
U.C.S. (MPa)	134.2	148.01	171.7	195.38	108.54	148.01
Young's modulus (GPa)	82.59	80.64	81.43	73.35	70.22	66.99
Poisson's ratio	0.21	0.21	0.16	0.17	0.16	0.14

Three off-reef sites were selected of which only two gave reliable results(Figure 12).

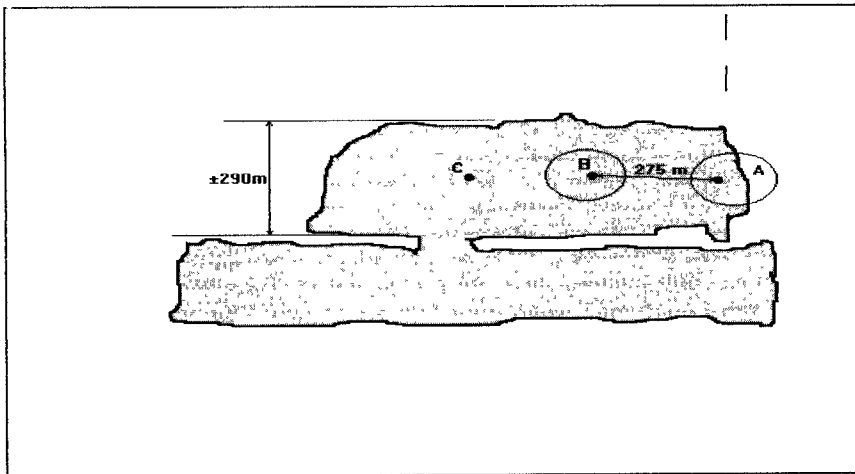


Figure 12. Three stress measuring sites below the reef plane at Kloof Gold mine

Table 5 *In-situ* triaxial measurements results using the *Doorstopper* technique. Results in MPa.

	$s_1$	$s_2$	$s_3$	$t_{xy}$	$t_{xz}$	$t_{yz}$
Site A	53,4	37,7	30,3	-7,09	-3,18	-0,32
Site B	31,9	20,8	13,9	-1,70	-3,70	1,57
Site C	<i>Unsuccessful measurements/poor core recovery</i>					

As measurements of site B were done below a stoped out reef one would expect extremely low stresses at this point, in view of the fact that it is a *de-stressed* area. An *in-situ* major principal stress of 31,9 MPa is more common in virgin ground at depths of approximately 1170 metres, not at 17 metres below a void. The results suggest that closure has occurred in the back area of the stope which results in the regeneration of stresses, an idea which can be supported by numerical modelling

MINSIM-D models were used to back analyse the stress measurements in the field. The objective of this work was:

To assess the behaviour of the rock mass for different values of the elastic modulus E=70 GPa; E=50 GPa; E=40 GPa; E=32 GPa, E=20 GPa

To assess the rock mass behaviour for different values of stope width up to 120 cm

To assess the behaviour of the rock mass for combinations of the above

To determine which of the above conditions more closely reproduces the in-situ stress results.

Use the calibrated model to ultimately analyse the effect on the pillar

### Numerical Modelling Results

The relevant conclusions from the modelling exercise are:

Using the commonly accepted rock characteristic elastic parameters for the Witwatersrand quartzites (E=70 GPa,  $\nu = 0.2$ ,  $\theta = 30^\circ$ ,  $\rho = 2700 \text{ kg/m}^3$ ) and a k-ratio of 0.5, total closure of the 290 m mined out span between stabilising pillars can not be predicted elastically in the longwall.

For a given value of the stope width of 110 cm, contact of the footwall and hangingwall is predicted elastically. This however will only occur if the elastic modulus is down graded to 40 GPa or less.

The zone of first contact within one grid element ( $\pm 5 \times 5 \text{ m}^2$  real size area) occurs at an roughly central *point* of the mined-out span, almost vertically coincident with the exact position where the *in-situ* triaxial stresses were obtained.

Table 6. Good agreement between the modelled results and the measured in-situ results from Site B

	<b>S<sub>1</sub></b>	<b>S<sub>2</sub></b>	<b>S<sub>3</sub></b>
Measured <i>in-situ</i>	<b>31,9</b>	<b>20,8</b>	<b>13,9</b>
Model Run-4	<b>31,9</b>	<b>18,8</b>	<b>13,1</b>

The implications of down-grading the modulus were also tested in the model. It was determined, for instance, that the value of APS on the pillar is about 6% lower if the modulus is reduced by about 47% .

Table 7. Table summarising the effect of reducing Young's Modulus on average pillar stress

<b>Elastic Modulus (GPa)</b>	<b>"E" reduction from 60 GPa</b>	<b>APS (MPa)</b>	<b>APS reduction from 152 MPa</b>
<b>60</b>	<b>0</b>	<b>152</b>	<b>0</b>
<b>32</b>	<b>47%</b>	<b>143</b>	<b>6%</b>
<b>10</b>	<b>83%</b>	<b>92</b>	<b>39%</b>

The average pillar stress in the stabilising pillars reduces because some of the load of the overlaying strata is transmitted through the areas that have closed in the back areas of the stope. The reduction in APS in the stabilising pillar suggests that the pillar may become more stable after a certain amount of mining which permits total closure to occur. The amount of APS reduction after total closure of the surrounding stopes is a parameter which may need to be considered in a design criteria. Further work to support this objective need to be undertaken.

## Seismic Monitoring

### Western Deep Levels

#### Calibration of the PSS

The importance of good quality data from the seismic networks was recognised and therefore much effort has gone into configuring the two PSS networks. A total of 39 development blasts occurring at known locations in a footwall development on the down dip side of the stabilising pillar were recorded. Both P- and S-wave arrival times were picked as accurately as possible for each of these events. Using this information, travel-time versus radial distance from the source to receiver were plotted and the apparent velocities were calculated. The resulting mean velocities were:

$$v_p = 5850 \text{ m/s and } v_s = 3700 \text{ m/s.}$$

When applying these velocities to the location of other events recorded by the PSS the error in location was reduced.

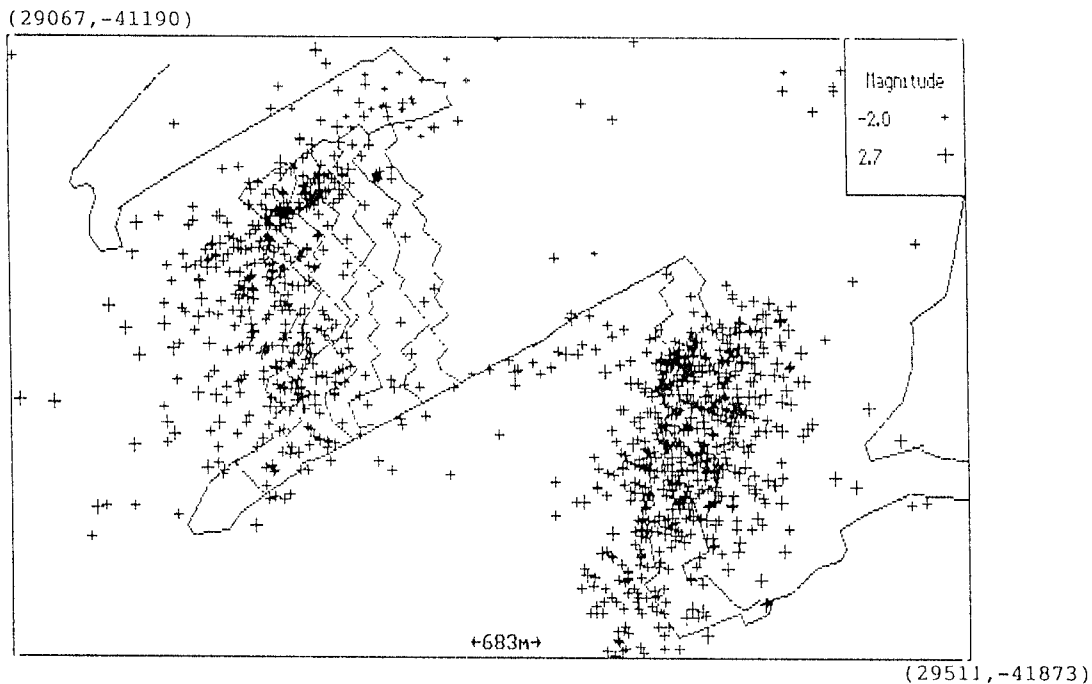


Figure 13. Plan showing the seismic pattern at the Western Deep Levels pillar site

#### Analysis of the recorded seismicity

The vast majority of events recorded by the PSS are associated with stoping activities as is evident in Figure 13. Figure 14 shows a time section through the stope; it can be seen that there is a tendency for the events to migrate with time from east to west as the face advances. Figure 15 shows the events recorded during the six month period 1/6/94 to 30/11/94. Comparing the distribution for the two longwalls shown it can be seen that the seismicity associated with the upper longwall, which is protected by a stabilising pillar, tends to be ahead of the face and less concentrated spatially. In contrast, the seismicity associated with the lower stope, which is not

protected by a stabilising pillar, is more concentrated spatially and the majority of events locate behind the face. A similar observation was made during mining with pillars at E.R.P.M..

The largest recorded event in the region of interest located in the hangingwall on the down dip edge of the pillar and had a magnitude of 2.7. This event resulted in damage in a nearby footwall development. There was no preceding or following increase in seismic activity in the vicinity of the event in question.

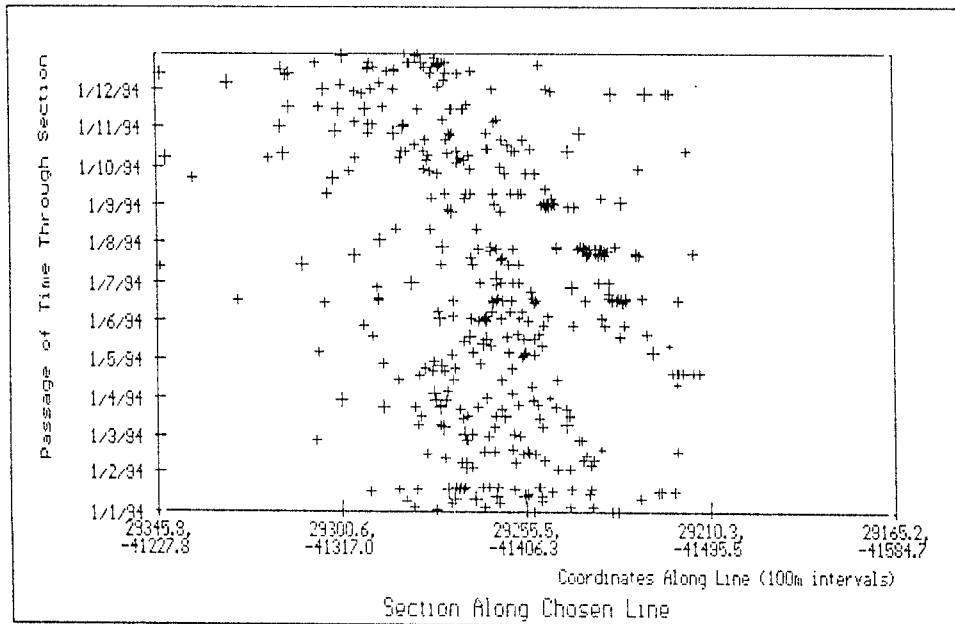


Figure 14. A depth section of the seismic events shown in Figure 13

On average, the stress drop associated with events within the pillar and in its immediate vicinity is higher than that for events in the stope region. A higher stress drop is indicative of the softness of the pillar and increased stresses in the rock, thus this phenomenon is not unexpected.

At the eastern end of the pillar there is a band of seismicity which coincides with the pillar narrowing from a width of 50m to 35m (Figure 13). An explanation for this concentration of seismicity has not yet been established.

#### Seismic Refraction Survey

A seismic refraction survey was carried out along a portion of the stabilising pillar on the down dip side. The survey line extended for 18m and a refraction profile was recorded in both the forward and reverse directions. Geophones were attached to the tunnel sidewall at 1.5m intervals and a 2 kg mallet was used as an energy source. The resultant signals were recorded using a notebook computer in conjunction with a 12 channel seismograph.

Interpretation of the results yielded a two layer situation with the first layer being between 0.5 and 0.6m thick with a velocity of 1140m/s and the second layer having a velocity of 4700m/s. The second layer is greater than 6m thick, in order to determine the thickness of this layer more

accurately a longer spread and a more energetic source are needed. From these results it can be concluded that at least the outer 6.5m of the pillar are heavily fractured.

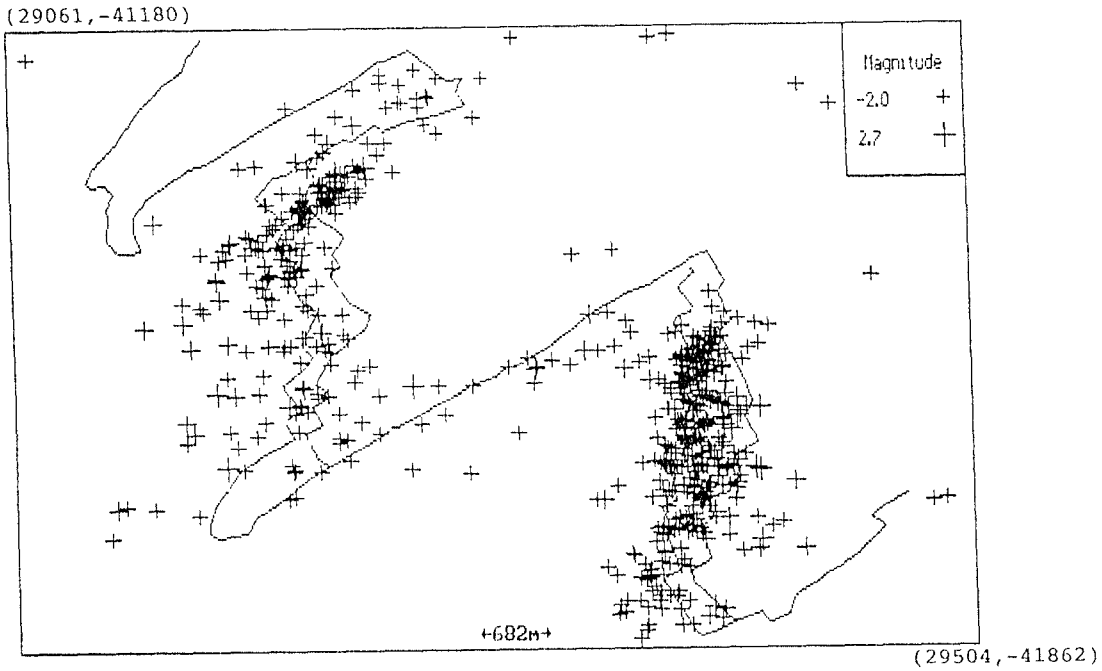


Figure 15. Seismicity recorded during the time period 1/6/94 to 30/11/94 at Western Deep Levels

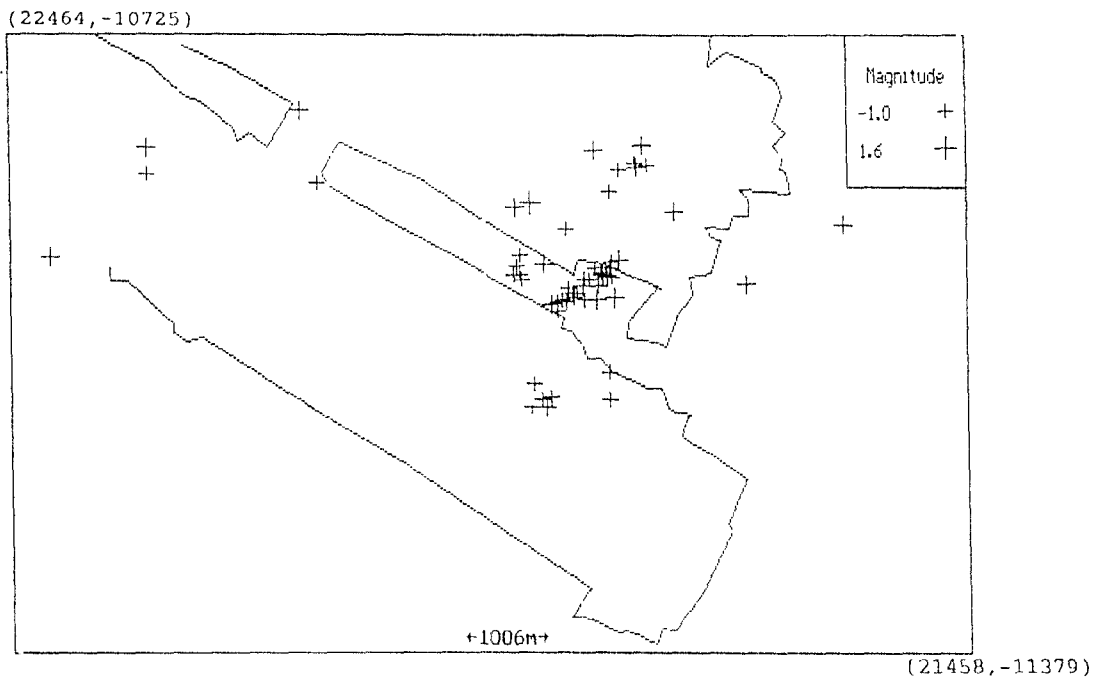


Figure 16. A plan of the seismicity patterns at Kloof gold mine

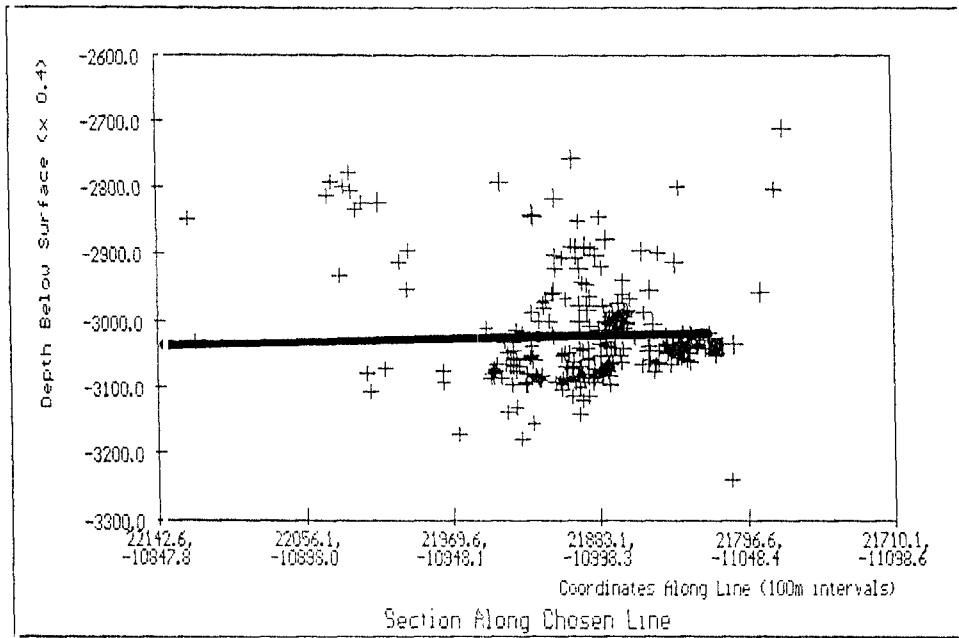


Figure 17. A depth section of the seismic events recorded at Kloof Gold Mine and shown in Figure 16

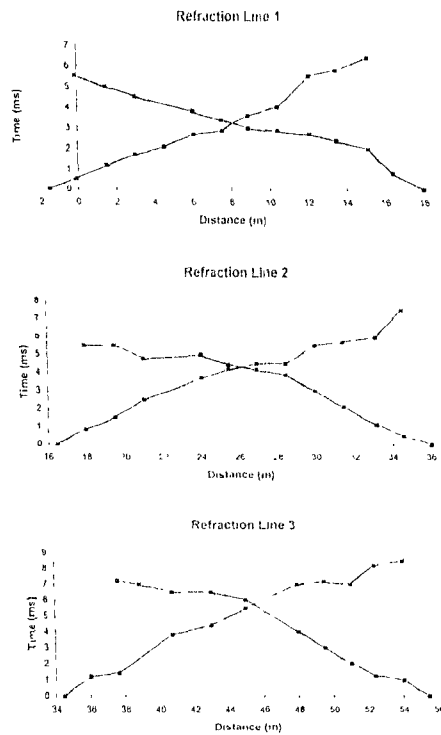


Figure 18. Six distance-time curves obtained from the first and last spreads of the refraction survey

## Kloof Gold Mine

### Analysis of Seismic Data

Figure 16 shows the seismic events recorded by the PSS on Kloof in the vicinity of the stabilising pillar. In contrast to the situation at W.D.L., there is practically no seismicity associated with face advance, although in the past the longwall on the up-dip side of the pillar did experience seismicity. The lower longwall has moved far ahead of the upper longwall and therefore the seismicity along these faces is expected to be low. The up-dip longwall spans 3 levels whereas that on the lower side spans only 2 levels. These longwalls are being mined using the breast technique, in contrast to W.D.L. where north siding is being practised in our region of interest.

Figure 17 is a depth section along the length of the pillar showing event locations over the past 2½ years.

### Seismic Refraction Survey

A seismic refraction survey along a footwall development was carried out. The footwall comprised Elsburg Quartzites with a seismic velocity for the intact rock of approximately 5800m/s.

The tunnel surveyed is located at a depth of 2956 m below surface. It passes below a stoped out region and then transects the foundations of the stabilising pillar being monitored. The survey line extended for 56m and was split into 3 spreads of approximately 18m. A refraction profile for each spread was recorded in both the forward and reverse direction. Geophones were attached to the tunnel sidewall at 1.5m intervals and a 2 kg mallet was used as an energy source. The resultant signals were recorded using a notebook computer in conjunction with a 12 channel seismograph.

A total of 6 seismic refraction surveys were recorded. First break times were identified on the recorded seismograms and distance-time curves for each spread were plotted. Figure 18 shows the six distance-time curves obtained.

From the two seismograms depicted in Figure 19 it can be seen that the frequency content of the received signal is greatly diminished in the second record. This is a consequence of the fracturing of the rock where it is subjected to very high stresses due to the stabilising pillar.

Curve fitting and interpreting for a two layer situation yielded the model depicted in Figure 20. It appears that the outermost layer, where the rock is very highly fractured, increases from a thickness of 0.4m at the start of the survey line to 3.3m at the end of the survey line. As the



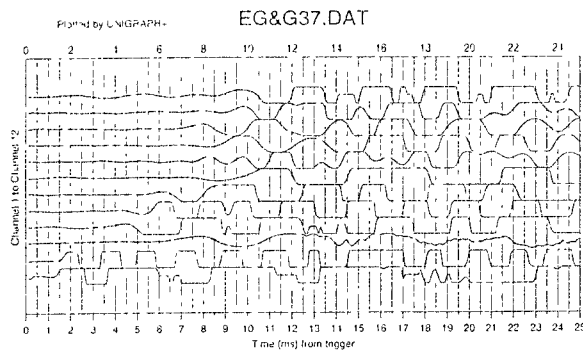
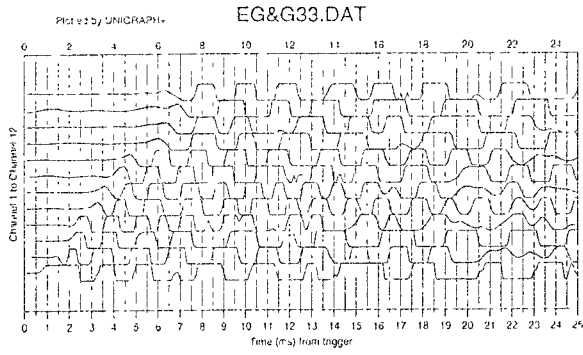


Figure 19. Forward seismograms for both the first and last spreads of the refraction exercise

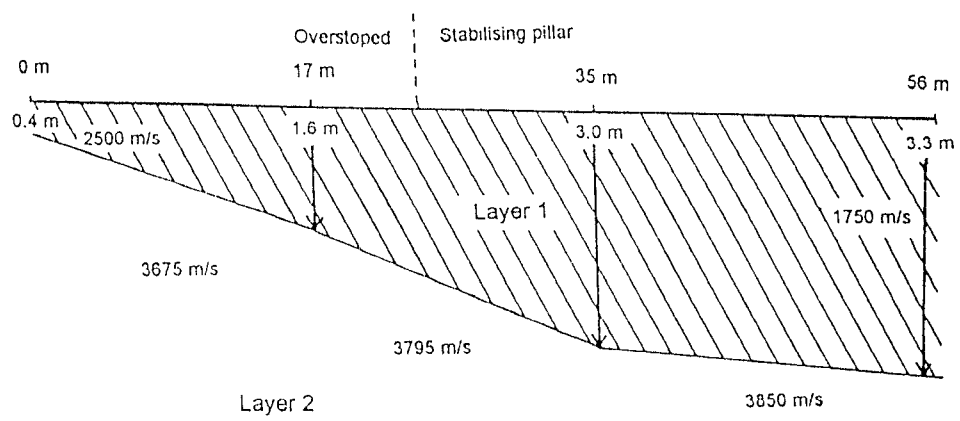


Figure 20. Model obtained from curve fitting, assuming a two layer situation

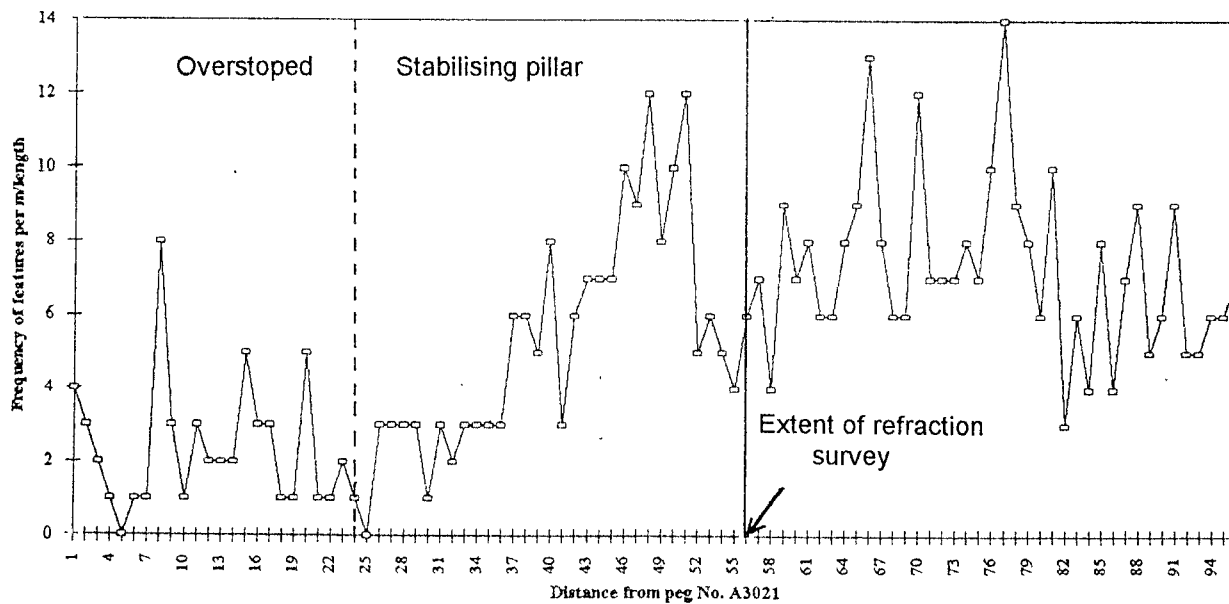


Figure 21. Plot of fractures mapped in the tunnel in which the refraction survey was conducted

thickness of this layer increases, the seismic velocity associated with it decreases. Both phenomena are a direct consequence of the stress regime in the area. Figure 21 shows a plot of the results of a fracture mapping exercise along the tunnel. The increase in the thickness of the outer layer and the decrease in seismic velocity described above coincide with the increase in fracture frequency. In contrast to the above results, the velocity associated with the second layer increases towards the pillar. This reverse trend can also be explained in terms of the stress regime; it is conceivable that the higher stresses below the pillar could confine the fractured rock thus enabling it to exhibit an increased seismic velocity.

### CONCRETE PILLARS

The trial concrete block underground was cast in June 1994. This was later than planned. However, there were severe logistical problems associated with transportation of some 80 pallets of bagcrete from the shaft station to the site of the trial concrete block. There were also delays associated with finding a contractor.

The instrumentation layout in the concrete block is shown in Figure 22. No significant stress levels or closures were recorded initially as a result of the mining sequence, which entails up dip mining. The current stope face is some 30m away on strike.

An 150 mm triaxial cell has been built to test large aggregate concrete or backfill mixed with aggregate samples. The cell is now in routine use and a programme of testing is in progress. The standard sleeve material used in normal backfill tests was not capable of providing a seal against the confining oil. Unlike backfill, cavities are formed in concrete, which, if near the surface of the specimen, collapse under confining pressure. The sleeve is then pushed by the oil into the cavity, inducing tearing if a polyurethane sleeve material is used. Two 3 mm thick rubber sleeves were eventually tried, with success.

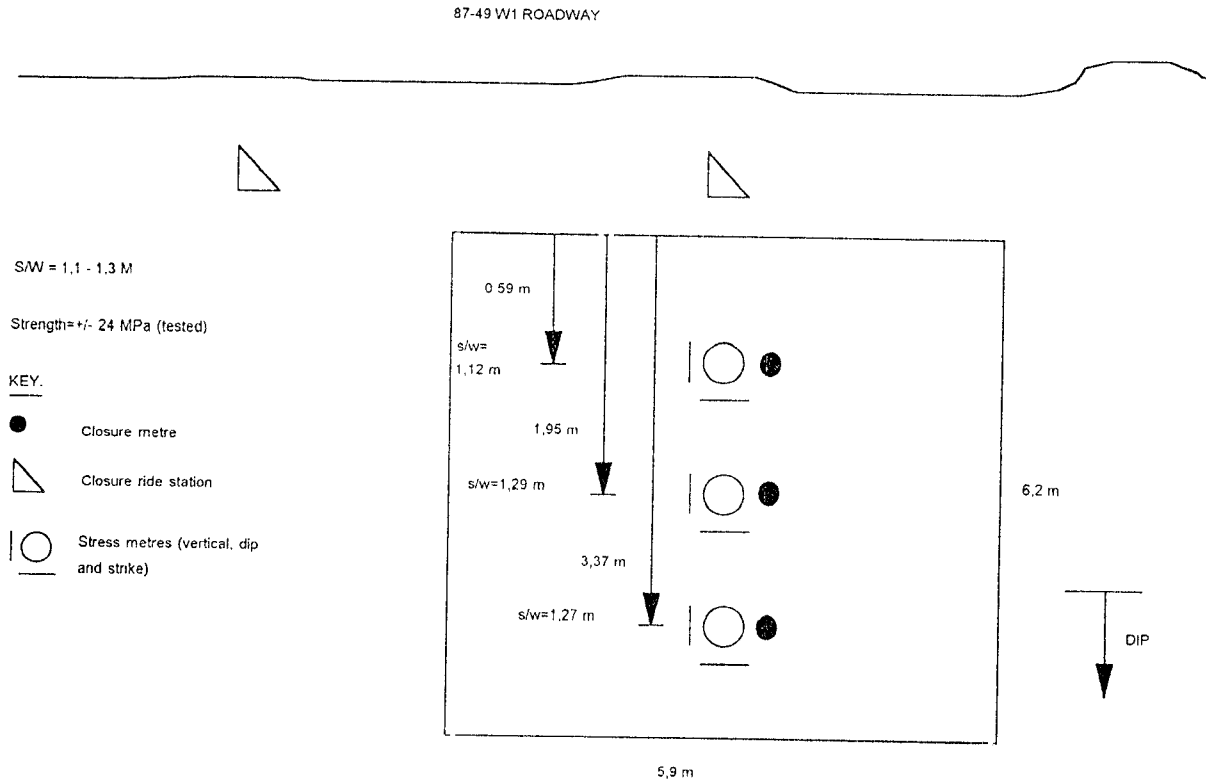


Figure 22. Layout of instrumentation inside the concrete paddock

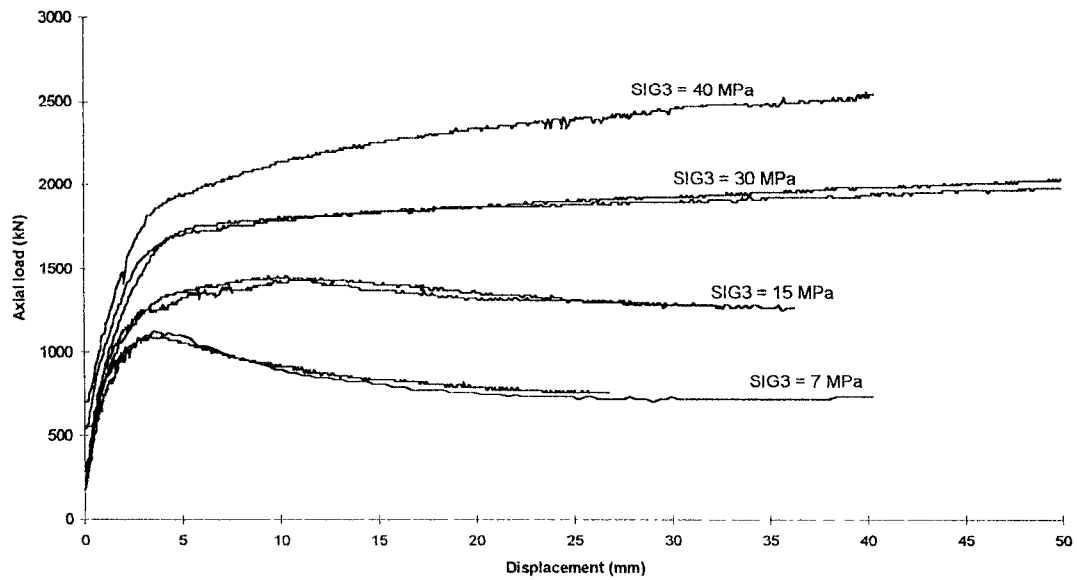


Figure 23. Behaviour of concrete tested at confining pressures of 7 to 40 MPa

The first suite of tests consisted of a “dry” concrete mix. The next suite will consist of a “wet” mix. A “suite” of tests refers to a series of tests at various confining pressures to obtain the complete confining pressure stress-strain behaviour of a given concrete mix. The results are used to

obtain strain softening parameters for back analysis of the concrete pillar behaviour by means of numerical modelling. The concrete mix has proved to be stronger than expected and to have a higher Young's Modulus than expected. The Young's Modulus is in the order of 18 GPa, almost double what was expected. This has significant benefit for regional support in that volumetric closure is inhibited.

The concrete shows stable post-failure behaviour at confining pressures of 7 and 40 MPa (Figure 23). However, when a confining pressure of 30 MPa is applied, the concrete yields but does not fail under axial loading. This is an extremely important result, showing that in the confined compression conditions in the middle of an *in situ* pillar with a large aspect ratio, the concrete will continue to accept increasing load.

Inelastic modelling of concrete and rock stabilising pillars was completed in January 1994. The model compared the behaviour of reef to concrete strike stabilising pillars. The criteria used were hangingwall stability, closure and average pillar stress. The concrete pillar was found to have the same performance as the reef pillar in the aspects of closure and average pillar stress, but was found to lead to more stable hangingwall conditions as a result of a lesser punching effect compared to that of the reef pillar. The modelling of concrete pillars compared to reef pillars shows that the regional benefits of reef pillars are not lost as a result of using concrete pillars. In fact, more stable hangingwall conditions are obtained in the case of concrete pillars due to a reduced punching effect at the edge of the pillar. These results were presented in the form of a paper, submitted in January 1994, and published in June 1994, at the 1st North American Rock Mechanics Symposium (York, 1994).

#### VOLSIM

VOLSIM is a 2 dimensional plane strain finite difference program which has been developed by CSIR Mining Technology to model deformations around openings underground. The program uses fast Fourier transforms which speed up calculations and inaccuracies in calculations. Pre- and post- processors were designed to set up mining geometries simply and quickly and to analyse the results of the numerical processing. The program is capable of producing excellent waveforms which may be analysed by PSS seismic software.

VOLSIM was benchmarked against other finite difference and boundary element programs. Quasi-static runs were three time faster than the other programs. The model is able to solve stope-fault intersecting geometries but copes better with non intersecting configurations. VOLSIM does, however, not give the correct crack convergence solution for large cracks relative to the model boundaries. It was also found to have large memory requirements. The ability of VOLSIM to simulate failure in the rock mass in large regional models is important for rock mechanics advancement.

Work on VOLSIM has been discontinued, but a scaled-down version of the previously proposed 3-dimensional VOLSIM is being investigated, specifically to study pillar failure.

#### BACK AREA CAVING

Back area caving has been successfully used as a mining method on a number of mines in the past. Currently, cave mining is only extensively used at Hartebeestfontein gold mine where it yields very favourable results. Work is being carried out with the assistance of this mine to gain a better understanding of the cave mining method, its benefits and conditions suitable for its application. Significant progress has been made in understanding the caving mechanism, in quantifying caving behaviour and benefits in the application of numerical modelling, to understanding the benefits of caving.

It appears that geology and the highly laminated hangingwall strata play a significant role in the caving mechanism. Thus the friction and cohesion between such layers is more important than the strength of the layers themselves. Dilation ahead of the face translates to significant horizontal hangingwall movement into the stoped out area. Arnold (1993) describes this phenomenon as "plastic" flow and notes the time dependence of such behaviour.

Good caving relies on the correct level of stope face support. Support type, spacing and resistance need to be "tuned" to achieve a good cave. Too low a support resistance can lead to loss of horizontal stress in face hangingwall and likelihood of premature fall-out. Too great a support resistance clamps movement on hangingwall strata and tends to inhibit cave formation. There is some evidence of extension fractures steepening if good caving is achieved, yielding a more stable hangingwall configuration (Figure 24).

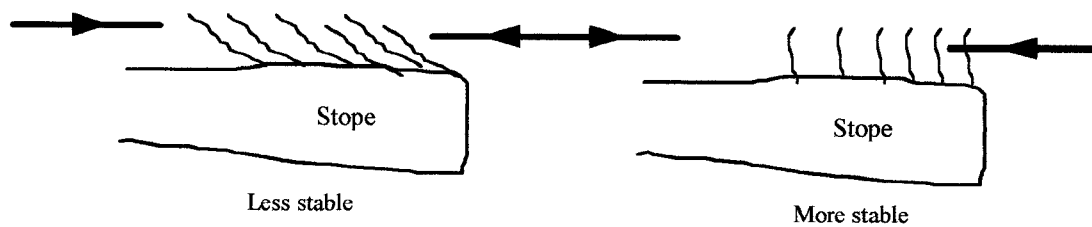


Figure 24. Good caving increases hangingwall stability.

Visual observations show that caving initially takes place with about 400 mm fallout and eventually increases to 2 - 3 m fall out. Hangingwall dilation has been measured taking place above 16 m in the hangingwall.

Despite much quantified information on stope face and hangingwall behaviour detailed in the cited references, little is known about the behaviour of the cave material itself. For this reason, pack load cells were adapted to measure the stress regeneration in the cave. Figure 25 shows the configuration of the 4 flat jacks used.

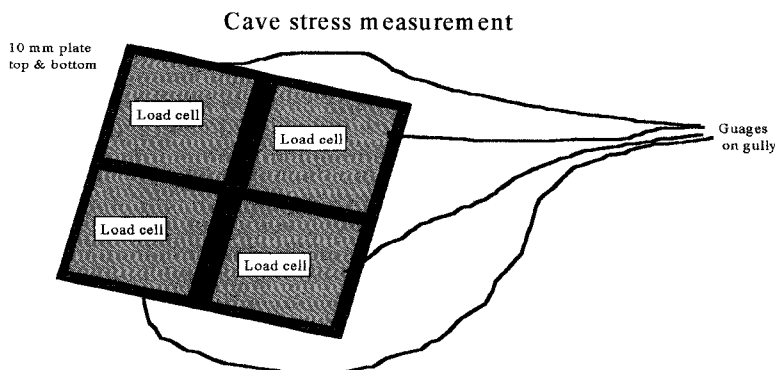


Figure 25. Cave stress measuring cell.

High stress regeneration in the cave material has been measured at three points in two caving panels. Despite some cells losing pressure/failing, the remaining cells continued to increase with advancing faces to 3 - 4 MPa (Figure 26), and even to 7 MPa in one case.

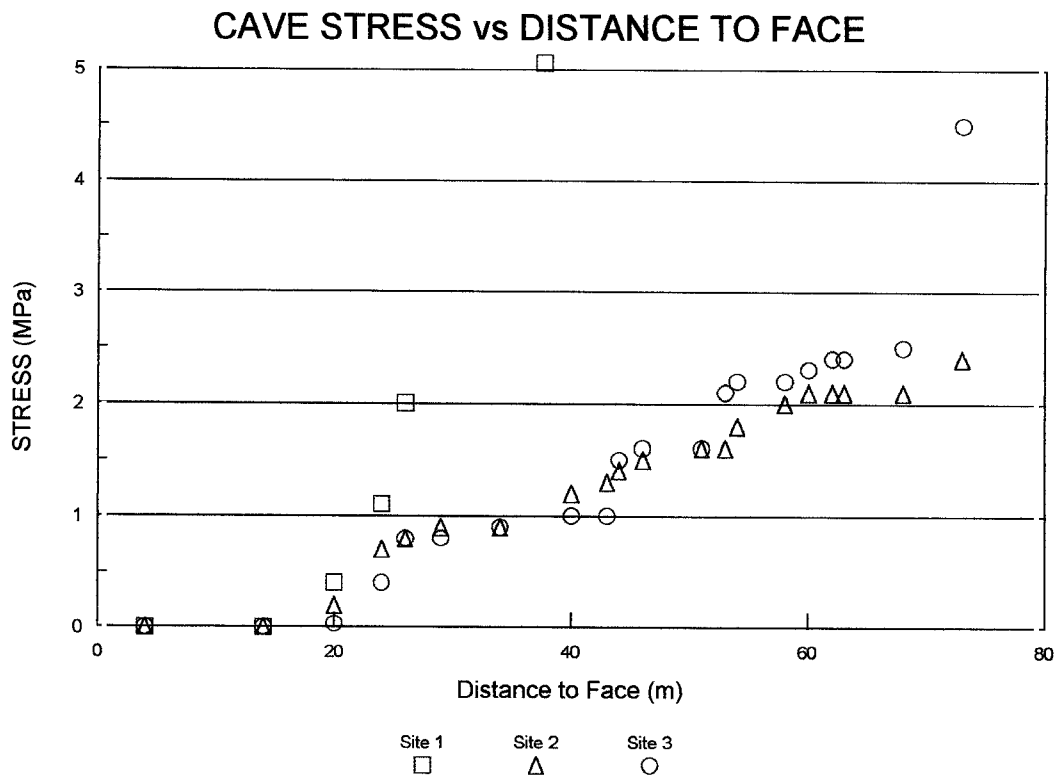


Figure 26. Stress regeneration in cave material.

One set of cells in an area of uniform cave showed very rapid stress increase at only 20 m from the face. Two other sets in areas with a more irregular cave increased much more slowly. Significantly, these stresses continued to increase after mining had ceased until access to instruments became impossible. Two additional sets of cells have been calibrated and have been installed in a suitable site at the deeper levels in the mine. Though no readings are available, this additional information will better quantify, and serve to validate findings to date. These high stresses are significant as they indicate that the cave material carries a significant load - in some ways similar to backfill.

The modelling carried out using UDEC has made good progress in qualitatively matching the conceptual model and observations described above. DIGS modelling of the cave mechanism has recently started - to complement the UDEC modelling. The UDEC modelling has given good qualitative results of the cave itself and of hangingwall/support interaction, but has not proved suitable for modelling the caving mechanism.

A very simple DIGS model has been set up comprising:

- symmetric stope and 9 mining steps
- four parting planes (0.6, 1.2, 2.4 & 3.6 m in the hangingwall)
- observations indicate that movement on hangingwall layers occurs and is significant
- parting plane properties have been varied (pre & post movement, cohesion, friction & dilation)

The different hangingwall parting plane slip areas can be identified with cohesion ranging from 0 to 10 MPa. This also increases the amount of stope closure by 32%. Further sensitivity studies will be carried out.

Some of the conclusions drawn from this work to date are:

The support benefit of a good cave are indicated by the high cave stresses regenerated.

The viability of the cave mining method in tabular deposits is highly dependant on the suitability of the geological conditions of the hangingwall of the stope.

Support requirements in caving need to be "tuned" to local conditions to achieve maximum benefit

A continuous and moderate rate of face advance is required for successful cave mining.

## ERR AND ESS CRITERIA

The first aspect of the work in this area was to carry out a series of back analyses to assess the relevance of ERR and ESS criteria for predicting seismic risk which actually occurred. Areas were chosen from all the mining districts and these were modelled with MINSIM-D. It was clear that it is not always true that the two criteria predict the same risk for the same layout.

Particularly the use of ESS should be considered where there are any geological anomalies present in the vicinity of the mining. It is apparent from the studies that the incremental values of ESS and not the total ESS is important in considering the risk of any layout. Often the total ESS associated with geological features in a particular layout shows no significant change with further mining. However, the ESS change associated with one particular feature. However, it should be noted that an increase in incremental ESS may not immediately result in an increased seismic risk but may indicate a risk in the future with additional mining.

The Evander mining area was also assessed even though it is known to have low seismicity. The idea was that such a study would assist in establishing the lower threshold of ESS values at which seismicity may be expected to begin. The vales of ERR and ESS were predictably low in this area given the low percentage extraction of this mining region and the depth of the deposit. The low ERR and ESS,  $3 \text{ MJ/m}^2$  and  $< 5 \text{ MPa}$  respectively, account for the absence of seismicity in this mining district.

The concept of area ESS (AESS) was introduced to describe the ESS in the most unfavourable direction in a plane above or intersecting a mining plane. For the analysis in this project a plane 30 m into the hangingwall was chosen as representative of the rock mass condition. This technique was then applied to some case studies. Total AESS was used in the initial phase and then incremental AESS was used. The incremental AESS shows a reasonable correlation with the cumulative seismic moment in some studies. The incremental AESS has to be normalised by the amount of mining which has taken place. A refinement on the AESS is the ubiquitous joint ESS (UJESS) which still uses benchmark sheets 30 m into the hangingwall but if there is a known joint set of reoccurring weakness at a particular orientation the ESS is calculated along that plane.

## OVERALL CONCLUSIONS

Although this is a project being addressed on a broad spectrum of issues the progress in each are has been significant and is their own right have contributed to a better understanding of the deep mine layout problem. Of great importance is the way in which each area has been able to contribute to the advancement in the other areas producing a synergy. Although far from complete the first two years of the project has laid a good foundation for the continuation of the work in this area.

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