

6.4.3 Empirical Design Rules

Empirical design rules generally require only very limited rock mass data, and are pitched at a fairly conservative level. This is so because of the large number of geotechnical environments from which they were derived, and for which they need to be applicable. Such design rules will however certainly not cater for localised abnormal ground conditions. General design rules for normal support systems frequently used within the mining industry are:

Length of tendon = $0.5 \times \text{span or height of excavation in which they are to be installed.}$

Spacing of tendon = $0.5 \times \text{length of tendon.}$

Consideration should be given to increasing the length of anchors in poor ground conditions, or high stress environments such as tunnels close to pillars; and to incorporate fabric support systems in highly friable ground conditions or where seismic activity may be anticipated. It is a general requirement that the installation of all secondary support be carried out prior to the onset of any high induced stresses.

The application of these design rules should be carefully considered with regard to previous experience in the defined geotechnical environment.

6.4.4 Design Philosophy

The application of a design methodology for tunnel support systems for a given environment will be based on the availability and quality of geotechnical data. Thus for initial design estimations in a poorly understood rock mass environment, simple design rules as given in section 6.4.3 would be used. As increased information on the rock mass becomes available (though local experience may still be limited), rock mass classification systems as discussed in section 6.4.2 may be used. These will allow the design engineer to implement support systems which have been found to be successful in comparable rock mass environments. There is thus a higher level of confidence involved, but these designs will still be limited by the general applicability and assumptions of the classification system. It now becomes the responsibility of the design engineer to monitor these new excavations by observation and applicable instrumentation (Chapter 10.5.2) to assess the effectiveness of the initial design recommendations. Interpretation of the results and assessment of the experience gained will enable the engineer to gain a better understanding of the rock mass behaviour and a clearer mechanistic understanding of the interaction of the support system with the rock mass. This will provide the basis for conducting structural and mechanistic analyses as described in section 6.4.1 to refine the support design to ensure safe operation over the life of existing and future excavations. It is to this level of design and understanding of the rock mass to which the engineer needs to strive. At all levels of design, the engineer needs to select the most applicable methodology for the rock mass environment under consideration.

6.5 SPECIAL CONSIDERATIONS

6.5.1 Corrosion

Corrosion of the support elements can dramatically influence the operational life of an excavation, or its need for rehabilitation. Corrosion may be associated with a corrosive environment within the tunnel, such as in a return airway or dirty water dams; or to corrosive ground water within the rock mass, particularly associated with geological structures such as faults and dykes. In these environments consideration must be given to the use of special corrosion-resistant steel materials, or protective coatings. The use of shotcrete as a barrier may be considered, although this has been observed to have a detrimental influence on rock bolts and cables within the support system in a corrosive ground water environment.

It is important to evaluate the quality of support in old areas which are to be reworked; notably in shaft pillar extractions, where support may have been in place for decades.

6.5.2 Rehabilitation

Rehabilitation of tunnels may be required due to the deterioration, or failure, of the current support system caused by seismic activity, long term stress changes, time-dependent rock mass deformations, or corrosion. The rehabilitation of tunnels is thus usually conducted under extremely poor conditions due to the failure of the previous support system. Even if the previous support system appears intact, and rehabilitation is being conducted because of deterioration over time and not because of failure, it is prudent to assume that the previous support has been compromised and that the tunnel is effectively unsupported. The use of a high standard of temporary support during the rehabilitation procedure is thus an important requisite. Ground conditions may have deteriorated to the extent that it may not be possible to drill within the fractured peripheral rock mass. Under these conditions, consideration needs to be given to pre-shotcreting, and the use of high quality circular, or horseshoe, steel sets as longer term support. In severe situations, resin or cement grouting of the rock mass should be considered.

The rehabilitation of mesh and lacing should be minimised, as this 'bleeding' process reduces the confinement developed by the support system to control deeper rock mass failure and deformation. The removal of this material will probably initiate further deformation, and may eventually compromise the effectiveness of the tendon reinforcement and the overall stability of the tunnel.

6.5.3 Support Quality Control

The design of a support system is only as good as its installed performance. The design process should thus consider the applicability of the technology, the sensitivity of the system to installation procedures and the systems available to control the quality of the installation process and thus ultimately the performance of the support system.

Initial laboratory tests should be conducted on the elements of any new support system, under conditions which simulate their interaction with the rock mass and their anticipated in situ loading. This may involve axial and shear testing of tendons under static and dynamic loading, as individual units and with any associated grouting system. Also desirable are static and dynamic load tests of fabric support systems, with appropriate linkages to any tendon anchorage and interaction with the rock mass structure. An appropriate evaluation of the adhesion of support elements such as shotcrete or other membrane support systems should also be conducted. This type of testing programme should consider the degree of variability within the performance of the support system, both for a single batch and over time, so that this may be considered in the setting of appropriate factors of safety in the design process.

Quality control systems should also include the appropriate education of the installation crews, and supervision of the installation process, to ensure adherence to the correct procedures. Certain components of the support system such as grouts, resins, shotcrete etc, have a restricted shelf life which must be considered in the ordering, supply and utilisation of these products.

In situ testing is a vital component of the quality control process as this is the only true reflection of the performance characteristic of the elements of the support system in the installed rock mass environment. Testing procedures should be conducted on a regular basis and be appropriate to the mechanism of interaction between the support element and the rock mass. Thus, pull tests may be conducted on certain tendons in situ, samples of shotcrete taken for compressive or beam laboratory tests, and the thickness and adhesion of shotcrete measured in situ. Visual inspections should also be conducted of the quality of the elements of the support system prior to installation and during the installation process.

Due to the highly variable nature of the rock mass and the assumptions made during the design procedure, it is highly desirable to implement monitoring systems such as extensometers and closure pegs in order to verify the performance of the support system. Information gathered from this instrumentation programme may be used to gain further insight into the rock mass characteristics and refine the design procedure for suitable safe and cost-effective support selection.

The scale of the quality control programme and monitoring requirements, and thus the resources put into these programmes, should be a function of the criticality of the excavation to the operations of the mine, the safety of personnel, and the degree of uncertainty in, or sensitivity of, the performance of the support system in the given rock mass environment. Typical quality control programmes may test approximately 1-2% of the elements, but may increase up to 10% if problems in the system are detected. The limit of acceptability of the performance of the support system should be based on the allowance of variability considered by the factor of safety in the design process. If inadequate support is detected, then additional support measures need to be implemented.

LARGE CHAMBERS AND SHAFTS

7.1 INTRODUCTION

This chapter deals with the layout and support of large underground chambers, shafts, and ancillary excavations. The rock mechanics principles involved are in many respects similar to those governing tunnel layout and support, and Chapters 5 and 6 should be read in conjunction with the present material.

7.2 LARGE UNDERGROUND CHAMBERS

Large underground chambers housing hoists, pumps and cooling plant or water settlers and sumps have operating lives of several decades, and their development cost is high. Damage to equipment housed in these chambers inevitably leads to loss of production and increased operating costs, and the time required to excavate and equip them often has a bearing on the total time required to bring a mine into operation. Therefore, due care must be exercised when locating, designing, excavating and supporting chambers of this nature.

As each site for large chambers has very specific geotechnical conditions, much will depend on the ingenuity of the designers in dealing with the particular problems presented. The experienced rock engineer will often be able to exploit the positive aspects of a particular situation to reduce costs without prejudicing either safety or stability.

7.2.1 Siting of Major Excavations

Many potential problems can be solved at the time when the site of a new shaft system and the associated large chambers is selected. At this stage the schematic layout of the shaft (and sub shaft system, if any) should be available, together with the geotechnical aspects of the rock formations to be traversed (quantification of rock mass properties is discussed in Chapter 10.3.5). It is usually possible to adjust the vertical position and to rotate the planned axes of the excavations with a reasonable amount of freedom to minimize exposure to high stresses or to the less competent geological horizons or structures. The strategy for protecting the shaft and chambers should be in place (shaft pillar size, partial or complete reef removal - c.f. section 7.3.1), which will greatly influence the siting of all chambers within several hundred metres of the shaft-reef intersection.

Water-bearing strata and fissures should be avoided as far as possible, as these give rise

to additional costs during excavation and require protective measures for the installed equipment. There is also the possibility of dangerous gases being liberated.

Wherever possible, the intersection of large throw faults, dykes and sills within the chamber should be avoided, particularly if these are prone to seismic activity and/or have an associated abnormal tectonic stress field. This rule should not be blindly adhered to however, as some large dykes and losses of ground have been used to advantage in the past.

The position of chambers situated close to the reef horizon should not be finalized until a complete stress analysis has been made of the site, for the different stoping configurations during the life of the chamber. If necessary, consideration may have to be given to partial or complete removal of the shaft pillar on at least one horizon. The positioning of settlers and sumps above a reef which is likely to be exploited in the future should be avoided, because of the possibility of leakage. Here, the case for pillars and backfill as support for the stopes may be entertained. The intersection of potentially payable reefs by a major chamber should be avoided.

Finally when selecting the site and layout of the chambers, consideration should be given to the situation when final withdrawal from the mine is to be made. It is possible that changes in location could be made which would result in considerable savings and greatly improved safety at that time, at very little additional present cost.

7.2.2 Stress Environment and Effect of Subsequent Mining

Hand in hand with location is the current and future stress environment to which the excavation will be subject. Here, it is important that the rock engineer has the best possible knowledge of the stress regime prevailing at the time of excavation, and a good idea of what the field stresses induced by future stoping will be. For this, numerical modelling (using codes such as MINSIMW) should be carried out to estimate the field stresses and stress changes likely to influence the sites under consideration.

Hoist chambers of sub-vertical shafts are usually situated well above the reef horizon, and suffer little change to the ambient stress field. Therefore, the shape and support chosen for the chamber usually need only address the stress environment which exists at the time of excavation.

Hoist chambers of incline shafts, pump chambers, settlers and some refrigeration plant chambers are often located close to the reef horizon and may experience considerable changes in the field stresses. The present and worst-case future stress fields must be considered when selecting the shape and support of these chambers.

Caution must be exercised when siting chambers in overstoped positions close to reef, particularly if the stopes have been backfilled, since stress regeneration resulting from stope closure can lead to serious damage when the fractured rock, in which the chamber is located, is re-loaded.

All major chambers have tunnels and subsidiary chambers associated with them. Care must be exercised when locating these chambers to space them sufficiently far apart, to avoid interaction of excavation stress fields giving rise to excessive and complicated fracturing of the intervening rock mass. In high-stress environments, the '6-diameter'

centres separation rule (Chapter 5.4.2) may be followed in simple parallel geometries; and more complex situations can be evaluated by numerical modelling where necessary.

7.2.3 Shape and Size

Here the maxim of '*simple is good, small is better still*' applies. Complex layouts give rise to irregular shapes and to high stress concentrations. Not only are these difficult to excavate and support, but numerous problems arise if the field stresses increase.

Probably, the most important consideration when selecting the shape of a chamber is the *nature of the rock* in which the chamber is to be located.

In massive rock formations, such as dolomites, lavas and thickly bedded quartzites, it is relatively easy to excavate a shape directly suited to the equipment to be accommodated in the chamber. Computer programs are available which enable the shape and orientation to be refined to minimise and quantify the support requirements. Access prior to the excavation phase is necessary to map joints and other geological discontinuities, if the full benefit of these techniques is to be realized. This is seldom readily available, but for vulnerable excavations it would be prudent prior to final planning to provide exploration tunnels to expose the necessary detailed geotechnical information required for proper design.

Strongly bedded sedimentary rocks, such as quartzite, require the chamber orientation and shape to be selected to suit the dictates of the rock. Here the long axis of the chamber should preferably be oriented on or close to the strata dip direction (or, in massive strata, perpendicular to any pervasive jointing) to enhance safety during excavation, to reduce support costs, and to improve the finish. This is often possible with pump and refrigeration chambers and horizontal settlers and sumps. It is not always possible with hoist chambers, as their positioning is dictated by the shaft conveyance arrangements.

In high-stress environments where clear parting planes are not present, the hanging-wall should generally be arched with a smooth transition to the sidewalls. In highly bedded or jointed strata, the excavation shape should, however, allow for and conform to the natural planes of weakness (see Figure 5.3.2a).

Another important aspect of the design of chambers is to *minimize their width*. Narrow chambers require fewer development cuts, are intrinsically more stable, and are much easier and less expensive to support. A concept worth addressing is that of placing machinery components that are not mechanically coupled in positions that enable the width of the chamber to be reduced. A good example of this is the motor/generator set of a Ward Leonard hoist which can be placed in a standard width tunnel close to, but apart from, the hoist chamber. Here the additional expenditure on cabling is more than offset by cost savings in terms of excavated volume, as well as enhanced safety.

The chamber width can be further minimized by using independent crawl beams suspended from rock bolts secured in the hangingwall or individual lifting points. These systems can replace the gantry cranes so often employed, and which require complex shaped shoulders and heavy reinforcing or additional steel columns as support because of the heavy fracturing that occurs in the sidewall in deep mining conditions. Gantry cranes, although more flexible than individual crawl beams, increase the width of the chamber by at least 2 m. Here, the specific requirements of lifting equip-

ment need to be carefully considered before making final selections.

The advent of efficient conical settlers in the 1960s eliminated the need for horizontal settlers. Some mines followed this by employing vertical sumps as well. In both these cases the amount of chamber roof requiring support is minimized.

Machinery foundations in chambers likely to experience serious increases in field stress need to be isolated from the chamber sidewalls to avoid possible tilting of the machinery due to sidewall movement. Where precise alignment of machinery is necessary, consideration should be given to *reinforcing the footwall* in such situations.

The excavation procedure must always be born in mind when planning a layout. Care should be taken to minimize the number of accessways, rockpasses and ancillary chambers required only to facilitate the excavation process.

7.2.4 Excavation Sequence

The excavation sequence of a large chamber and choice of its associated support are closely inter-related problems, with the size, shape and geotechnical setting of the final product dictating the combination selected. In this Chapter, the term **primary support** means tendons of sufficient length and density to support and stabilize the exposed rock-wall of the cavern in its *final* dimensions; **secondary support** (which generally involves shorter but more densely spaced tendons, plus surface fabric cover) is designed to provide stabilization and prevention of surface unraveling, both during excavation to protect the development crews, and in the long term. [Note that these definitions, which are in common use, differ significantly from those used in tunnel support (Chapter 6); in which 'primary' signified 'short-term' or 'immediate' support, and 'secondary' signified 'long-term' support able to accommodate stress changes during the lifetime of the tunnel.]

In chambers where rock fracturing is slight, excavation sequencing is less important than in deep excavations where the control of fracturing and subsequent support assumes a dominant role. In all situations though, the need to ensure that the excavated portion of the chamber is safe, stable and properly supported at all times is paramount, with such factors as speed of excavation and equipment selection being secondary. The size, shape and direction of each excavation phase must be selected with stability in mind at all times. Ongoing monitoring throughout the development is desirable to identify potentially unstable blocks in good time.

(i) *Establishing the roof*

Because the roof of a completed chamber may be many metres above the final floor, it is usually excavated first so that rockwall dressing and support installation can be carried out effectively. Here, the excavation must be properly supported at all times with the primary support tendons installed as early as possible and certainly long before the final span is established. Preferably the lining, where necessary on the roof and final sidewalls, should be applied before the body of the chamber is excavated, thus maximizing safety. The two methods most commonly employed both enable the early installation of support tendons. As will be seen later, the ambient stress field prevailing at the time of excavation can be the deciding factor as to which method is selected.

Method 1. In narrower chambers where the full width can be effectively blasted as

a straight face, the final roof can be exposed as a single cut. Here the permanent primary and secondary support tendons must be installed and grouted at the face as soon as the tendon position has reached a distance of about 1 m from the face. This distance is governed by the ability to drill the support holes properly.

Method 2. In wide chambers it is better to excavate the roof in a series of cuts (Figure 7.2.1). Where the chamber is long, this is best approached as a single end never wider than twice the length of the secondary support tendons selected and advancing in a down dip direction. During stage 1 (Figure 7.2.1a) the secondary support tendons are installed to pattern with the primary support tendons' position marked for reference. The primary support tendons are installed when convenient (Figure 7.2.1b stage 2) but prior to the excavation of the second cut. The repositioning or installation of additional support tendons, to accommodate potential keyblocks exposed during excavation, should take place at this stage. [Analytical methods or computer programs are available to assist in the design of this additional support.] The second cut (Figure 7.2.1c stage 3) is then taken wide enough to expose the position for the next line of primary support tendons. This exposure is thus initially supported using the secondary support tendons, followed by the primary support tendons required in the second cut (stage 4). The third and any subsequent cuts can then be made in the same manner (Figure 7.2.1d).

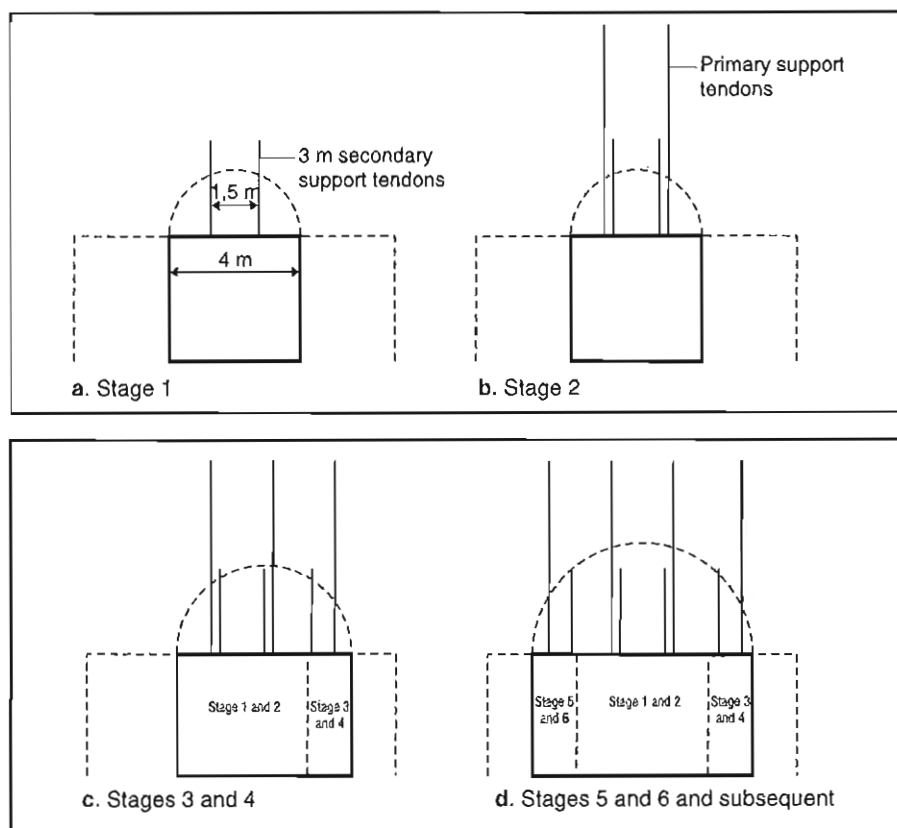


Figure 7.2.1 Preferred sequence for supporting and excavating a wide excavation
a: Stage 1 (Conventional 4 m wide tunnel). **b:** Stage 2 (Installation of primary support). **c:** Stages 3 & 4. **d:** Stages 5 & 6.

It will be noted that at all stages of both methods, the load capacity of the installed permanent support tendons must be capable of carrying the deadweight of rock within the potential collapse zones of the partially completed excavations (dashed in Figure 7.2.1).

The lining of the roof should be installed at a distance from the face where it is not damaged by the blast. If the roof rock is friable, it may be necessary to apply a layer of shotcrete or other fabric to the roof as a temporary measure.

Advancing in a down-dip direction enables the bedding planes to be reinforced by support tendons prior to them daylighting into the excavation.

High-stress situations require special treatment, as extensive stress fracturing will occur during excavation. The selected shape and sequence must ensure that the stress fractures are positioned and oriented in the most convenient manner to facilitate support installation and minimize surface unravelling. Under conditions where the major principal field stress is nearly vertical, use can be made of the fact that horizontal tunnels reach stability when they have scaled to an elliptical cross-section (major axis horizontal). Thus to achieve the most stable shape and avoid excessive fracturing, the roof should be excavated as a single horizontal ellipse (see Figure 7.2.2a). An arched roof conforming to this shape is advisable and will result in considerably reduced support costs. In this case only the upper 2/3 of the ellipse need be excavated in the initial cut. The lower 1/3 will be fractured anyway, and can be slipped out later. Here again, advancing in the down dip direction will facilitate early clamping of the bedding planes along which the stress fractures in the hanging wall will tend to be located. The support sequence outlined in Method 1 applies best here (Figure 7.2.2b); the use of Method 2 with its rectangular sections would, under these conditions, result in an unstable and unnecessarily fractured roof which would be difficult to support.

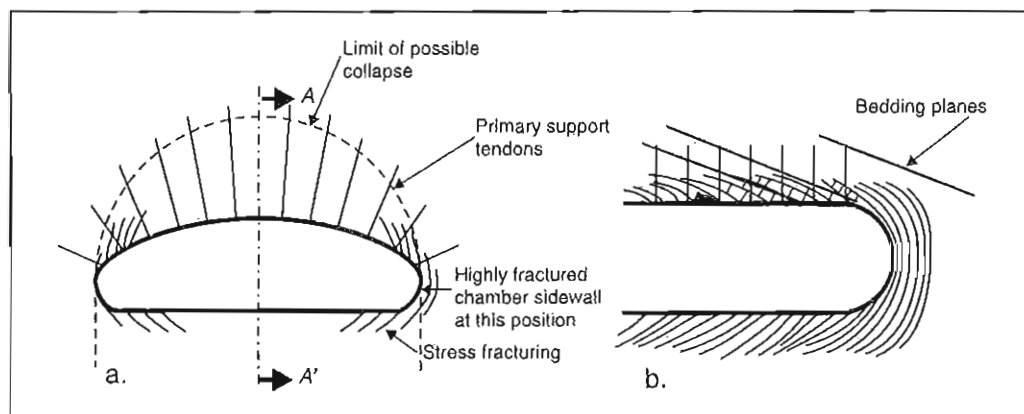


Figure 7.2.2 High stress excavation. a: End view. b: Section on AA'.

(ii) Excavating the body

Once the chamber roof is secured, attention can be paid to excavating the body of the chamber. Here the sequence is of lesser importance, except that effort should be made to expose the final sidewall from the top down, and as late in the cycle as possible. This facilitates the use of cushion ('smoothwall') blasting techniques, thereby minimizing blast damage and sidewall unravelling.

When excavating in very high stress situations, stress fracturing becomes such a dominating factor that the entire excavation shape and sequence is dictated by it. The limited number of excavations made under these conditions indicate that, wherever possible, the full chamber cross section should be excavated simultaneously. This is difficult in the case of large hoist chambers, for example, but is certainly possible in the case of smaller pump and fridge chambers and main belt transfer tunnels at the bottom of deep mines.

(iii) *Some case studies*

1. Hoist chamber in brittle quartzite with prominent bedding planes. Here the situation was complicated by layers of quartzite up to 2 m thick which were known to pose severe safety problems, because of a tendency for the sidewalls to slab prodigiously in existing tunnels. The hoist chamber was relatively small (20 m x 15 m). In this case, the decision was made to commence excavation by first establishing the updip top corner of the chamber by a tunnel driven on strike, and installing the secondary tendon support in the roof and sidewall (Figure 7.2.3a). The roof was then exposed by slipping out the downdip side of the tunnel, carrying a bedding plane as the hangingwall (Figure 7.2.3b). The permanent chamber roof was then established by a line of postsplit holes and the primary support tendons installed as described in Method 1 (Figure 7.2.3c). This procedure was repeated until the entire chamber roof was established and supported, after which the chamber body was excavated by first establishing and securing the sidewalls.

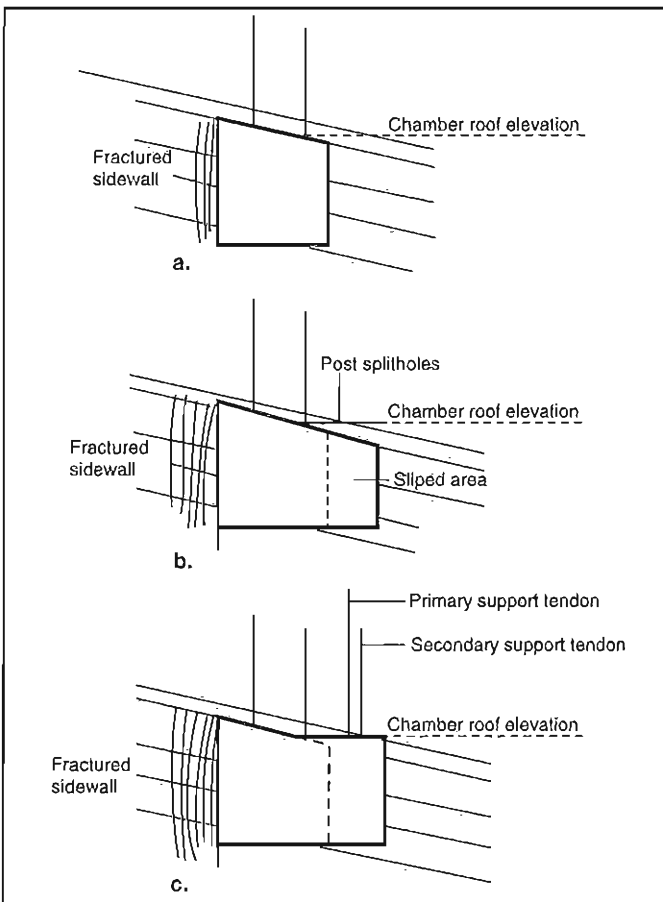


Figure 7.2.3
Stages in excavation of hoist chamber in bedded ground.

2. Conical settlers and vertical cylindrical clear water sumps in weak argillaceous quartzite. These excavations both had 10 m diameter circular roofs. The support comprised a ring of mechanically anchored cable anchors as primary tendons, augmented by 3 m long by 19 mm diameter rock studs as secondary support. The position was secured by a 3 m wide-end driven diametrically across the chamber and supported by the secondary tendons. The centre was then slipped out to beyond the central primary support positions, using the secondary support tendons as support. After these primary support tendons had been installed and grouted, the final slipping was completed and supported, and the roof was lined.

3. Belt tunnel at depth. The twin belts, for transfer of rock from rock silos to shaft measuring bins, are generally accommodated in a single tunnel of rectangular cross-section. Because severe stress fracturing up to 3m into the sidewalls had been seen in tunnels of near square cross section in the vicinity, it was decided to excavate the belt tunnel with an elliptical cross-section, and to excavate to the final dimensions in a single pass. This was achieved successfully, and subsequent sidewall inspection holes revealed a fracture zone depth of only 0.4 m. The tunnel was supported with grouted rebar shepherds crooks, meshing and lacing.

7.2.5 Blasting

Cushion ('smooth') blasting techniques should be used to provide the final roof and sidewall finish of chambers, irrespective of the ambient stress conditions. Although blast hole barrels will soon disappear due to surface unravelling in high stress conditions, blast damage is virtually eliminated.

Flydirt damage to the chamber roof lining can be greatly reduced by using shock tube or electronic detonators when excavating the body of the chamber.

As in tunnel development, attention to drilling accuracy, correct spacing and charging/stemming of perimeter holes, and attainment of sequential firing will more than repay the effort involved.

7.2.6 Support

Design methodologies, and the properties of types of available support units and systems were described in Chapter 6. The support systems employed in service excavations need to be designed with three specific objectives in mind, namely:

- to facilitate the ease and timeous installation of support during the excavation phase, and thereby to stabilize and make safe the zone of fractured rock around the chamber during its development;
- to accommodate later changes to the fracture zone and resulting dilation caused by creep, or by changes in the stress field in the vicinity of the chamber, or by possible seismicity;
- to consolidate rock surfaces of the chamber to avoid long-term unravelling and consequent injury of persons, damage to machinery and possible destruction of the support system integrity.

(i) *Roof support*

In stress regimes where the major principal field stress is nearly vertical, it is a generally accepted rule of thumb that the volume of rock in the roof requiring support is

enclosed by a semi-circle of diameter equal to the width of the excavation (Figure 7.2.4). In a vertical excavation of circular section, this equates to a hemispherical dome. This assumption is based on observations of roof collapses that have occurred in inadequately supported excavations in poorish quality rock, and most probably errs on the side of conservatism. [More precise, though more technically complex methods are currently under development, involving prior geotechnical analysis together with numerical modelling.]

It is good practice to support the total weight of rock in the fracture zone by the primary support tendons anchored beyond the limit of the potential unstable zone. It should be noted that the manufacturers' stipulated working load should not be exceeded; that is, the ultimate strength of the tendons needs to be somewhat greater than the working load, giving a built-in factor of safety.

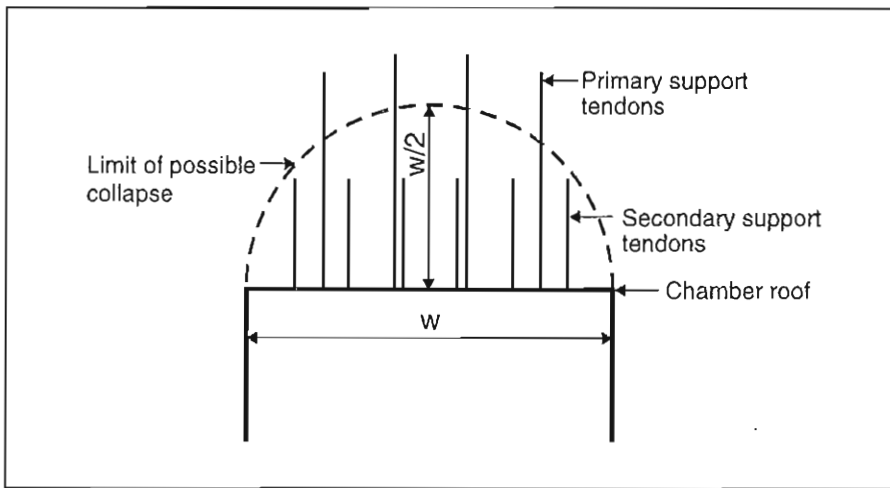


Figure 7.2.4 Primary and secondary support above a large excavation.

Long tendons are either coupled steel rods or multistrand 'cable'. Two forms of anchoring are used. **Mechanical anchors** are usually employed with tendons where the pre-tensioning load requirement does not exceed about 300 kN. **Grout anchors** which require several metres of cable to be grouted a number of days before tensioning, often have a working load of 1500 kN or more. These are usually employed in major excavations such as hoist chambers with horizontal roofs. All support tendons are full column grouted to preserve the pre-tensioning should the collar bearing plate or anchoring fail, to reduce corrosion effects, and to bond the surrounding rock to the support. Effective grouting of long anchors is difficult, particularly if the roof is fractured or ground water is present, and should be done by specialists.

In long-life excavations such as hoist chambers where support rehabilitation above machinery would be awkward and costly, end-anchored 'cable' tendons coated with grease and encased in plastic sleeves are often used to prevent corrosion. Here the load-bearing potential of the grouted bond is lost and particular attention must be paid to the bearing plate to ensure that prestressing is maintained.

The selection of tendon to be employed depends on the load to be supported, the nature of the likely fracturing, yield requirements, the roof excavation cycle and the time available. These supports are expensive and it is extremely important that selection and installation be done with care. Generally, mechanical anchors are quicker to install than grout anchors but because of their relatively low bearing capacity, many more are required.

The spacing of *primary* tendons is usually decided by the working load of the tendon, the load to be supported, and the size of the initial excavation establishing the working roof. Primary support tendons are often spaced 3 m or more apart, and therefore apply little support to the surface of the intervening roof. Here the secondary support tendons, which are usually full column grouted pretensioned rock bolts, are deployed. As a rough guide, these bolts are positioned at one-half or less the spacing of the primary support tendons and are spaced one-half their own length apart. [More specific, but as yet not fully proven procedures to design support spacings, taking into account geological discontinuities and the need to ensure interaction between adjacent support units, were described in Chapter 6.]

The function of *secondary* support tendons is to reinforce the immediate roof between the primary support units. The load they bear must never be taken into account when designing the primary support tendon loads, unless the secondary support anchors extend beyond the limit of the potential unstable zone. They often serve as support for the roof of the tunnels used to establish the chamber roof, prior to installation of the primary tendon supports.

In this discussion, only a flat roof has so far been considered. If, however, the roof is arched (Figure 7.2.5), the weight of the annulus of rock to be supported is considerably less. As the roof line approaches the limit of the fracture zone and a natural rock arch develops across the span, the load to be carried by the support tendons diminishes until conventional rock bolts will suffice. This occurs when the annulus is less than 3 metres thick (Figure 7.2.5); and, provided the excavated height is 3 m or more, 3 m long x 19 mm diameter rock bolts may be used.

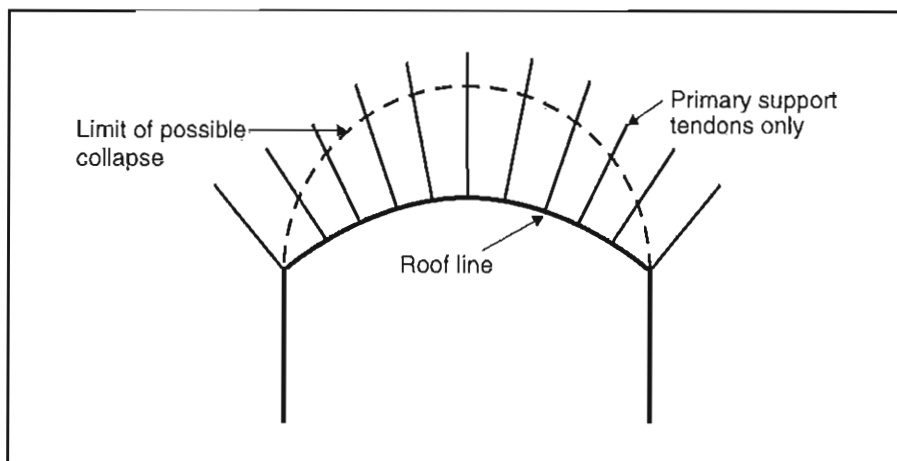


Figure 7.2.5 Reduced length of tendons above arched roof.

(ii) Sidewall support

The recommended length of the primary support is about one-half the height of the chamber wall, with the same spacing rule applied as for the hangingwall support. Shorter anchors are usually deployed between the primary anchors.

All tendons should be grouted in place. However, the amount of sidewall dilation that can be expected needs to be assessed and, if excessive, catered for by utilizing tendons with appropriate yieldability. This can be accomplished by not grouting the full length of long cable tendons by a predetermined amount (special anti-corrosion measures being necessary), or by using special high-yield tendon types.

(iii) Footwall support

In excavations housing critical machinery, consideration should be given to installing adequate reinforcement to the *footwall*. Similar standards to those used for sidewall support are appropriate.

(iv) Excavation lining

Various linings can be used, with selection depending on the function and finish required. A rough finish is usually acceptable. Concrete does not accommodate the deformations which often occur with stress changes, and is seldom used in modern mines. Galvanised diamond mesh, with an aperture of 50 to 100 mm, is popular as fabric between tendons. This, however, tends to sag between roof supports, is easily damaged by blasting during excavation of the chamber body and is difficult to shotcrete effectively. Weldmesh is probably the most suitable lining fabric, particularly if care has been taken with the wall finish when blasting. Apertures of 100 mm or more can be most effectively shotcreted.

Shotcrete provides an acceptable finish and minimises damage to the roof fabric during excavation of the chamber body. Fibre-reinforced shotcrete is preferred where significant deformation of the sidewall is expected.

(v) Quality control

The support quality control guidelines of Chapter 6.5.3 apply even more stringently to large service excavations. Spot *pull-tests* or other checks on up to 10 % of the installed support units should be carried out. Comprehensive *anti-corrosion* measures are most important. Finally, use of *monitoring* programmes in particularly sensitive chambers can pay dividends in anticipating problems caused by slow long-term rock mass movements, allowing timeous installation of additional support where necessary.

7.3 SHAFTS

Shafts are crucially important excavations, and many considerations govern their design and operation. Rock engineering principles are involved in shaft siting, in the design of shaft pillars or other shaft protection procedures, in aspects of sinking technique, in the layout and support of ancillary service excavations and orepasses, and in the extraction of shaft pillars.

7.3.1 Vertical Shaft Design

An important rock engineering consideration in the **siting** of shafts is that the reef intersection should, if at all possible, lie in geologically undisturbed ground. The common practice of siting shallow shaft pillars in fault losses should be avoided in deep mines, because of the hazards of slippage or seismic activity on the fault surfaces involved. If major seismically-active planes are in fact found to intersect a shaft pillar or the shaft itself, appropriate counter-measures need to be considered: substantial upgrading of nearby regional support, installation of an automatic hoist trip-out system, or even suspension of shaft steelwork either side of the fault intersection. [Displacements of up to 400 mm have occurred on such fault planes in association with large seismic events.]

Protection of a shaft against normal mining-induced strata deformations is traditionally accomplished by leaving a substantial **shaft pillar** of unmined ground around the shaft barrel. Occasionally the leaving of a shaft pillar may be avoided by sinking beyond a sub-outcrop, or by terminating the shaft above the reef horizon and cross-cutting through the troublesome reef intersection; but both of these options, while flexible in terms of opening-up imperfectly known (in terms of grade or geological disturbance) blocks of ground, involve inflated footwall development and tramming distances – Figure 7.3.1 – and are not widely used.

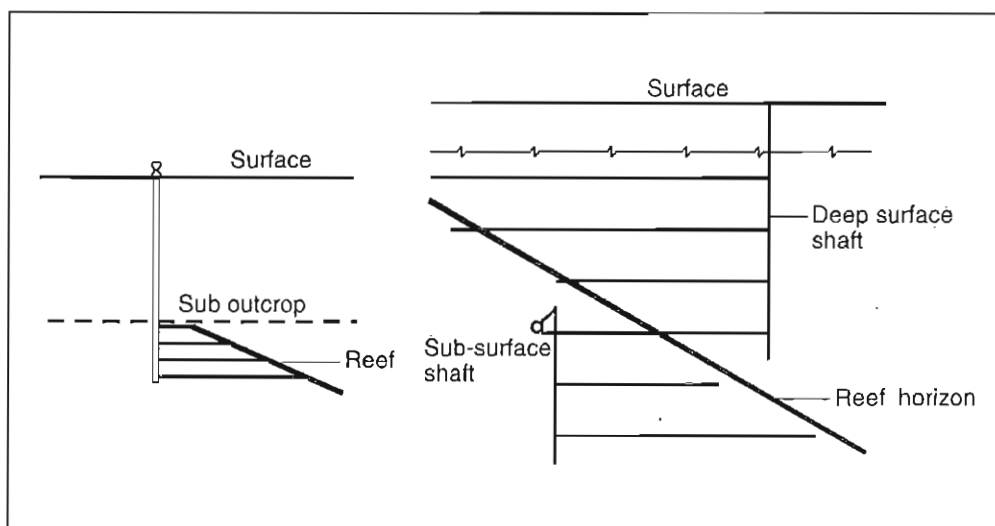


Figure 7.3.1 Avoiding a shaft pillar by a: Sinking beyond a sub-outcrop, or b: Crosscutting through and mining out the entire reef horizon

Conventional shaft pillars may be square or roughly circular in outline, and a basic design consideration is to determine a suitable radius R for the pillar. An old criterion (imported from shallow coal-mining practice in Europe, aimed at protecting surface headgear installations) was to make R one-fifth of the depth H to reef intersection. This rule-of-thumb led to a number of seriously under-designed pillars in some of the older South African gold mines. For depths H exceeding about 500 m, it

turned out that, while the pillars were satisfactorily carrying out their primary function of totally inhibiting tensile bed-separation deformations in the hangingwall, they were also acting as *stress-concentrators*. Once large spans had been mined, high mining-induced vertical compressive stresses began causing steelwork and lining damage in the shaft barrel, and equally seriously, in vital major service excavations near the centre of the shaft pillar.

Thus, a basic criterion for the design of shaft pillars in intermediate or deep mining conditions, is that the absolute vertical field stress in the centre of the pillar should not rise above a critical level denoted σ_{cr} . For good quality quartzites (having UCS $\sigma_c > 200$ MPa), σ_{cr} is about 100 MPa, beyond which RCF levels (Chapter 3.2.9) rise above 1.2 and the support of service excavations becomes increasingly difficult and expensive, and beyond which serious shaft sidewall and steelwork damage starts to manifest.

Figure 7.3.2a gives a worst-case (conservative) illustration of required shaft pillar radii R for given values of σ_{cr} as a function of depth H . It will be seen that as depth increases, the required pillar radius R begins to increase dramatically. Figure 7.3.2b illustrates this explicitly for the case $\sigma_{cr} = 100$ MPa, in which it is seen that a shaft pillar in fact becomes an increasingly non-viable option for depths in excess of about 3000 m. Figure 7.3.2c illustrates another problem with conventional shaft pillars: not only does their stress-concentration effect lead to enhanced vertical stresses (well above virgin conditions) at the centre of the pillar, but stresses toward the rim of the pillar rise to massively high values. Crosscuts and haulages exiting the shaft pillar can thereby begin to experience damage in the later stages of the life of the shaft and may require enhanced support or even over-stoping to restore satisfactory conditions. Mines with mature shaft pillars are well-advised to install suitable simple *monitoring* systems (Chapter 10.5.3) in the shaft near the reef intersection and in vital service excavations, to gain advance warning of stress-related problems and to facilitate appropriate remedial measures – steelwork adjustments, installation of additional support, overstopping, etc.

Figures 7.3.2 are conservatively plotted for a typical stoping width of 1 m, and assume a worst-case of total extraction around the pillar. In practice, fault-losses and other unpay areas may be known to be present and, thanks to planned regional support (stabilizing pillars and/or stiff backfill), the *effective* stoping width may be considerably lower than the nominal actual stoping width, leading to significant reductions in the required pillar radius R . Thus in practice, shaft pillar layouts are best designed with the aid of *numerical modelling* programmes (Chapter 11.4). End-of-life (worst-case) conditions should be modelled as realistically as possible, including all known geology and planned installed regional support. Field stresses should be evaluated at planned service excavation sites, and compared with σ_{cr} or RCF criteria. Estimates of compressive strains should be taken out along the shaft axis and compared with empirical strain limits of about 0.4 mm/m and 0.7 mm/m for normal shaft steelwork and shaft linings, respectively. A broad strategy for *ultimate shaft pillar extraction* should also best be set in place at this time.

Shaft pillars lock up ore reserves and clearly are best located in **low grade** or unpay ground. If payable reserves are present, these will usually be extracted only at the

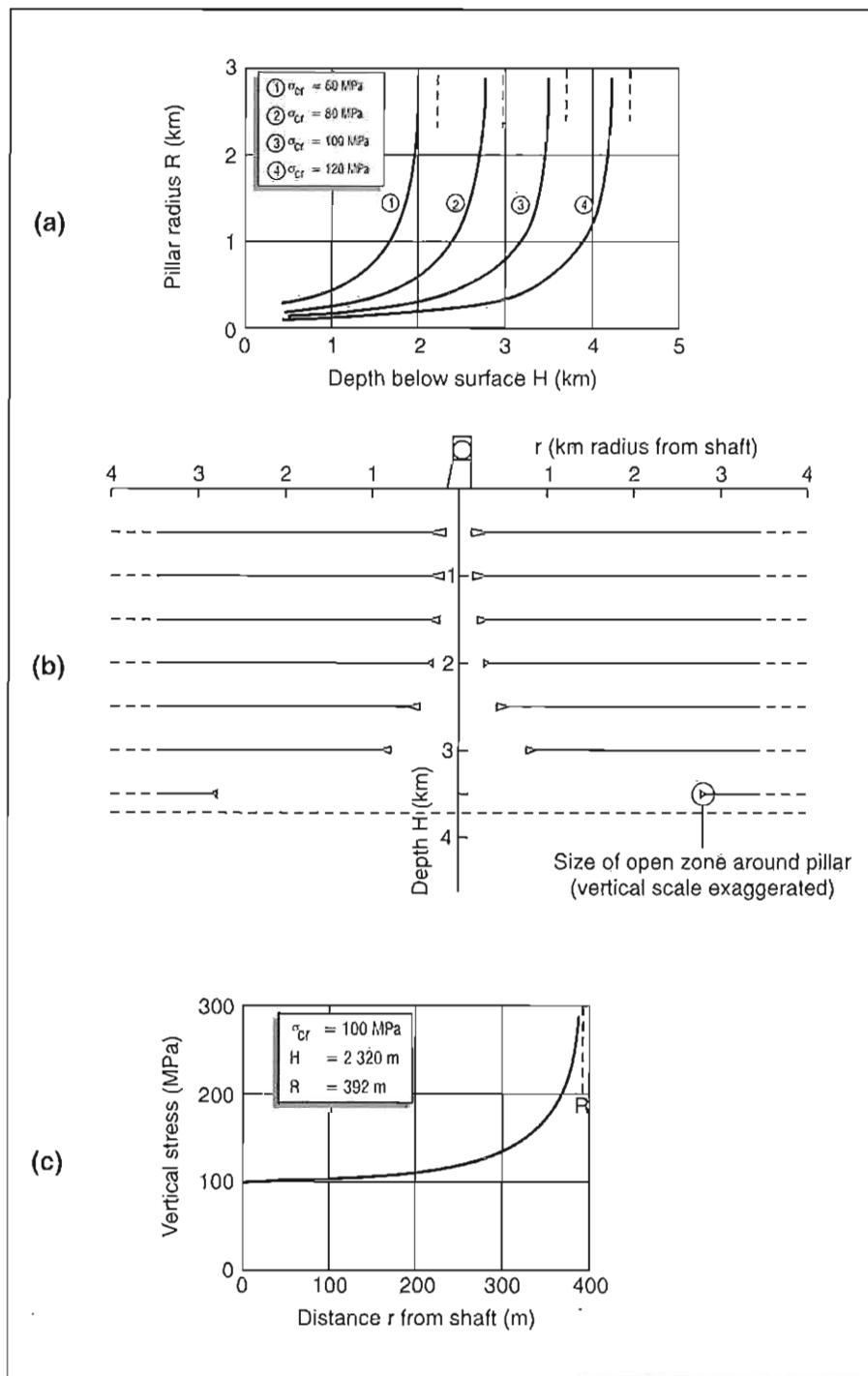


Figure 7.3.2 Mature shaft pillars [analyzed for $E=60$ GPa $\nu=0.2$ Eff.Sw=1m].
 (a) Required pillar radius R vs depth H , to limit central field stress to σ_{cr}
 (b) Cross section of required pillars at increasing depth, $\sigma_{cr}=100$ MPa
 (c) Typical stress distribution along radius of a shaft pillar.

end of the life of the shaft, when stress conditions are at their *worst*. Indeed, the exercise of late shaft pillar extraction is both hazardous and costly, and demands extremely careful planning – section 7.3.6.

Figure 7.3.2b clearly shows that (for good quality quartzites $\sigma_c = 200$ MPa, $\sigma_{cr} = 100$ MPa) for depths exceeding about 3500 m, the option of using a shaft pillar becomes totally untenable. The required pillar diameter *exceeds 5000 m*, and the field stresses away from the pillar centre, once extensive mining has taken place, rapidly build up to unacceptable levels (Figure 7.3.2c). At depths exceeding 3 700 m, the *virgin* vertical stress rises to above the σ_{cr} level of 100 MPa; this implies that the sidewall rock in service excavations and in the shaft itself becomes difficult and costly to support, even before any stoping has taken place.

Overstopping by early removal of the shaft-reef (reef around the shaft-reef intersection) becomes imperative. In deep mines, this procedure has been successfully implemented in a number of well-documented cases; and indeed, it can be an attractive option for depths as shallow as 1500 – 2000 m, for reasons which are listed in Figure 7.3.3.

Advantages of early shaft-reef removal

1. Immediate access to reef around the shaft, and greatly reduced early development and tramming distances.
2. Immediate, and permanent, overstoping protection to vital shaft installations.
3. Total avoidance of late shaft pillar extraction problems.

Potential problems with early shaft-reef removal

1. The shaft sustains immediate shortening and ride dislocations around the reef intersection (Figure 7.3.6), which become increasingly severe if a large (such as a cumulative multi-reef) stoping width, or a large dip angle, have to be catered for. These need to be absorbed immediately by appropriate suspension of shaft steelwork and services around the shaft-reef intersection.
2. The shaft-reef hangingwall strata can sustain tensile bed-separation movements which could lead to shaft lining damage, or early inrush of ground water before adequate pumping facilities are in place.

[These problems can however be ameliorated, or eliminated, by use of appropriate near-shaft regional support measures – use of backfill, pillars and/or delayed installation of permanent shaft steelwork/services. These issues are elaborated in section 7.3.5.]

Figure 7.3.3 The advantages and disadvantages of early shaft-reef removal.

Thus the strategic decision on whether to have a shaft pillar or to immediately remove the shaft-reef needs to be taken at the shaft design phase, with the following aspects to be taken into account:

i Depth of reef. The size of pillar required is a function of the critical field stress permissible and the depth of the reef - Figure 7.3.2. Deep and ultra-deep mines have little option but to stope the reef around the shaft if they are to avoid tying up an inordinately large percentage of their ore reserves in a shaft pillar.

ii Dip of the reef. The flatter the reef the less the horizontal deformation to be accommodated in the shaft upon removal of the shaft-reef – Figure 7.3.6. Generally, in past shaft pillar extractions, reef dips of less than 30° have been relatively tractable whereas dips of over 50° have given rise to significant difficulties.

iii Thickness of reef. The amount of vertical and horizontal displacement in the shaft is governed by the amount of convergence permitted – Figure 7.3.6. Therefore, a decision must be taken on the stoping width used and regional support to be installed. If necessary, some of the reef channel may be left unmined to reduce the effective stoping width near the shaft-reef intersection.

iv Number of reefs. The stoping out of one reef should eliminate any stress-related problems, but the stoping of more than one reef in close proximity will increase the potential shaft deformations considerably, if inadequate selective mining or regional support is employed in these horizons.

v Faults. Normal faults displacing the reef in or close to the shaft by tens of metres have been successfully handled in the past, though a fair amount of waste stoping was required to safeguard the shaft installations. Reverse faults can be more difficult particularly if associated tectonic stresses exist. Whenever possible the seismic history of faults should be established.

vi Igneous intrusions. Dykes and sills should present little problem provided that they are handled using good mining practice, particularly with respect to rockbursting.

vii Presence of ground water. Where there is a threat of large inflows of ground water, reef removal should be delayed until adequate means of handling excessive inflows through the shaft exist. Regional support should be sufficiently stiff to eliminate the chance of strata separation at the aquifer horizon.

viii Position of reef. When wishing to stop the reef during a shaft's working life, care must be taken to position the levels such that the shaft/reef intersection lies approximately midway between two stations, so as to avoid disrupting normal station operations. Alternatively, the abandonment of the station through which the reef passes has to be considered.

ix Type of shaft. Shafts equipped with brattice walls separating up and downcast compartments have a very good chance of severe leakage occurring; serious consideration should be given to employing twin shafts if early reef removal is required. Circular and elliptical monolithic concrete-lined shafts are the easiest handled; brick-lined and unlined rectangular shafts pose a constant threat of scaling/wedge failure, and short-circuiting of ventilation via strata separations which are difficult to detect and counter.

If the decision is taken for early removal of the shaft-reef, the shaft should best be sunk through **high grade** (at least payable, and preferably geologically undisturbed) ground. The procedures involved are discussed further in section 7.3.5.

7.3.2 Vertical Shaft Sinking and Development

During shaft **sinking**, sidewall support requirements will vary according to stress and

rock mass condition; from simple spot bolting, to pattern bolting with sheetmetal/weldmesh or shotcrete in heavily fractured ground. Stringent making-safe procedures need to be followed, and in severe conditions consideration may need to be given to installing the concrete lining only a few metres above the sinking face. Fault zone intersections may require stabilization by installation of long fully-grouted steel ropes. Cover drilling of overlapping sets of pre-cementation holes is a requirement at all depths for water ingress control. Geological mapping, supplemented with *geotechnical* (rock mass classification) studies at sites of all future service excavations, should be carried out during sinking.

Station cutting and other forms of development are usually carried out in parallel with sinking. For continued safety in the shaft, stations require immediate installation of support as in any other service excavation, following the guidelines of chapter 6 and of section 7.2. It is advisable to extend the station support for a short distance along the shaft barrel so as to reinforce the upper and lower brows of the station. Other major service excavations should be laid out and supported as described in section 7.2.

The **shaft-reef intersection** should best be positioned about midway between two stations. In the zone within about 30 m of the intersection, reinforcement of the concrete lining (with vertical hook bars and conventional mesh/lace plus shotcrete) is desirable in anticipation of ultimate shaft pillar extraction. At the intersection it is also good practice to interrupt the shaft lining and to pre-mine a 3 m ring of reef around the shaft, thereby considerably simplifying the task of later stoping out the shaft-reef without risk of damage to the shaft lining. The shaft brows here should be strengthened by substantial mesh and lacing, and the ring of mined-out reef durably supported.

7.3.3 Orepasses

Orepasses are often developed during shaft sinking. In deep mines, very severe spalling and 'self-mining' problems have manifested in operational orepasses, particularly when traversing weak horizons. The following precautions can mitigate some of these problems:

- i) An orepass should dip sub-parallel to the major field stress, at steep angles ($> 60^\circ$) to the horizontal, and should intersect weak strata as near perpendicular as possible. Usually this translates to the rule-of-thumb 'avoid orienting orepasses on strike'.
- ii) To minimize problems associated with impact and abrasion, individual legs of an orepass should not exceed about 70 m in length; though the use of funnel-shaped silos can relax this requirement to some extent.
- iii) To ensure that passes at the extremities of the system are not located too far from the shaft, the system may be located on both sides of the shaft. Nevertheless, some 'dog-legging' will usually be necessary. These dog-leg segments will break the rule of being perpendicular to the strata layering, and should therefore be located in strong rock horizons and be appropriately shaped and supported.

These issues are illustrated in Figure 7.3.4.

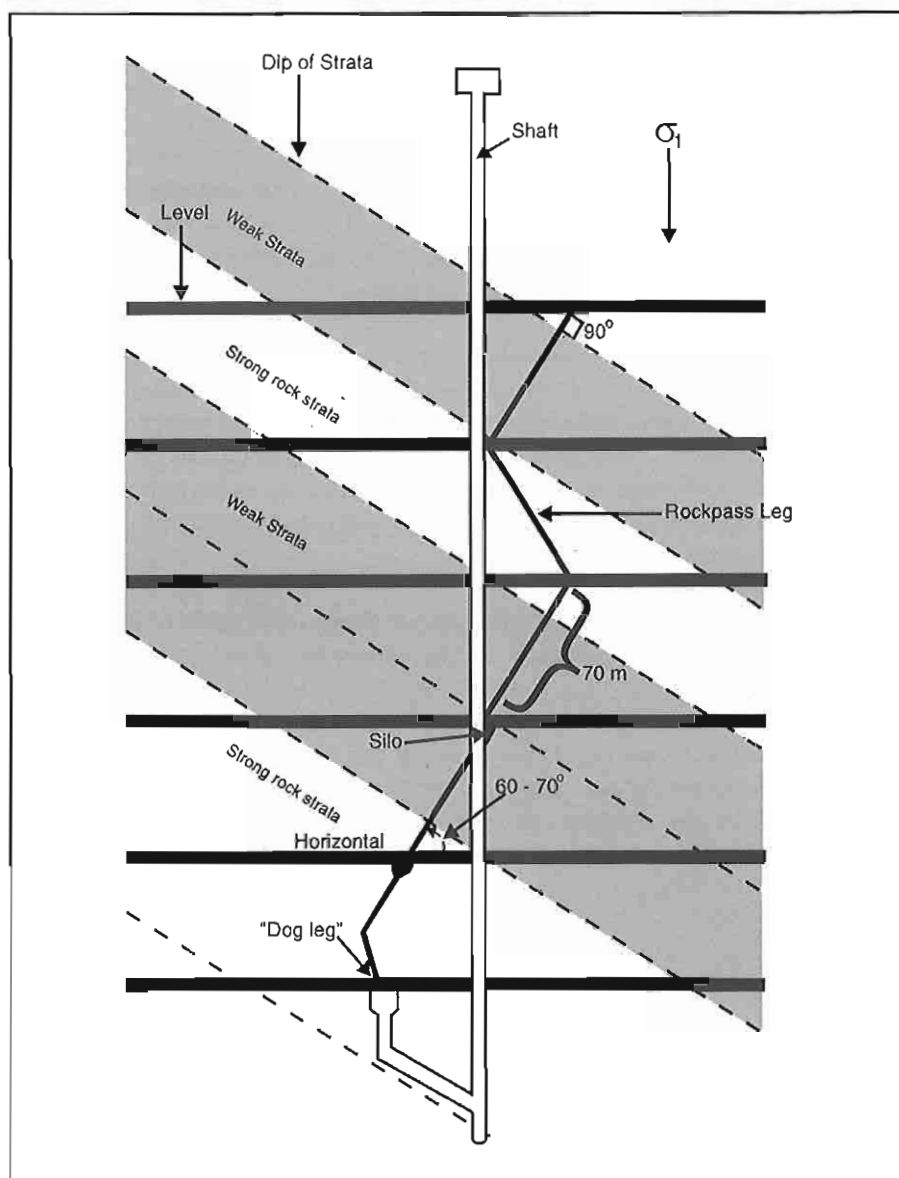


Figure 7.3.4 Orepass layout particularly suited to deep or ultra-deep mining

In orepass segments traversing *weak horizons* (shale, soft dyke etc.), substantial (> 10 diameter) clearances from neighbouring excavations should be arranged. An elliptical pre-enlargement of the orepass (major axis perpendicular to the major horizontal stress direction), followed by filling with abrasion-resistant concrete or shotcrete back to a circular outline, can minimize subsequent difficulties. A more cost-effective option, which is currently showing promise, is to create a thick abrasion-resistant circular shell, with conventional backfill introduced into the elliptical void – Figure 7.3.5. This backfill provides an inexpensive cushion to shock loading, and resists stress-induced spalling of the weak surrounding strata.

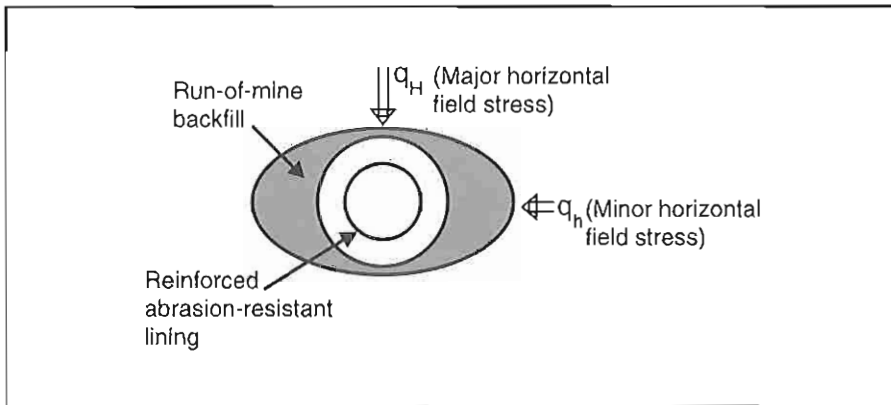


Figure 7.3.5 Excavated outline and lining suitable for orepass traversing weak strata, in an environment of unequal horizontal field stresses.

Raise-boring as a method of excavation offers advantages of speed and reduced damage to the rockwalls, and is well suited to shallow/medium mining situations. However, the potential disadvantages include

- the possibility of jamming, especially in deep, high field stress situations
- the circular cross-sectional shape, which may not be ideal where significant stress anisotropies exist (c.f. Figure 7.3.5)
- the inability to install immediate support, and thus to inhibit spalling and unravelling of the rockwalls – this can pose significant safety problems to support installation crews.

A deep-mining alternative is a ‘*bottom-up*’ drill and blast technique, for example making use of an Alimak machine. All supporting, both primary and secondary, can be done immediately after some safe advance. An added advantage is the opportunity offered for elliptically shaping the profile according to any field stress anisotropies present – Figure 7.3.5. A cheaper, though possibly slower, alternative, known colloquially as the ‘*flying banana*’ method, offers similar advantages. A cable, via a pilot hole from above, is used to winch up a platform which is bolted to the sidewall of the pass at convenient stages of advance. In both cases, it is necessary to protect the workers by means of a sturdy roof above the working platform.

Where pilot holes are used, these should be diamond drilled. A study of the recovered core will enable the rock engineer to design appropriate support and lining strategies for the various rock types to be traversed by the pass. Experience, moreover, has shown that *early effective support* pays dividends by reducing the need to embark on hazardous and extremely costly rehabilitations of passes that have deteriorated due to lack of appropriate support, or support installed too late. Keeping operational rockpasses full at all times offers the advantages of inhibiting deterioration of the rockwalls, by providing confinement and reduced impact shock. This desirable state will obviously very frequently conflict with production pressures, and so lining and support designs need to be based on sound experience and given a conservative bias.

7.3.4 Incline Shafts

Incline shafts are commonly seen in shallow operations, but are also still sometimes used in deep mining to open up regular blocks of ground (unfaulted with relatively constant dip). The major advantage of an incline shaft, is that it maximizes access to the orebody, notably for shallow angles of dip; but the hoisting efficiency and capacity are very significantly less than that provided by conventional vertical shafts, unless belt conveyors or other forms of transport are installed.

In shallow mines, incline shafts are often located on the reef horizon and, mainly for surface subsidence control, are sometimes protected by pillars of reef left intact on either side of the shaft. If the reef is non-planar or faulted, the incline shaft is best located off the reef horizon with no intact reef pillars in the vicinity.

Similarly in intermediate or deep mining, **immediate overstoping** of the shaft is desirable and is widely practiced. This permanently reduces the stress levels on the shaft, though significant long-term stress regenerations can still occur. Overstoping should be carried out over the entire length of the shaft even if initial production concentrates on shallower levels; if this is not done, the shaft will later encounter high abutment stresses at a point along its operational length. Stabilizing pillars, backfill ribs, remnants, or areas of low stoping width should not feature in the overstoped area immediately above or below the shaft, in order to avoid later regeneration of pockets of high stress or rockbursting which has occurred in the past in such situations.

In **siting** the shaft, incompetent strata should be avoided if at all possible; and in deep mines the distance to reef should not be less than about 40 m, in order to avoid encountering stope fracturing.

The **support** of an incline shaft has to take into account the large cross-sectional area of the excavation, and the fact that often its dip differs slightly from the average dip of the strata leading to the presence of thin wedges in the hangingwall. Standard tunnel support principles apply (Chapter 6), but particular attention should be given to the use of a tendon/mesh/lace/shotcrete system, which has been found to provide superior long-term support to that provided by monolithic concrete, particularly when overstoping is carried out.

7.3.5 Early Removal of the Shaft-Reef (Vertical Shafts)

Early removal of the shaft-reef has very significant short and long-term advantages – Figure 7.3.3 – but engineering means have to be found to counter the shaft dislocations that will occur, which according to Figure 7.3.6 can be quite large: of the order of 1 m or more depending on dip and the effective stoping width. Section 7.3.6 below describes these measures in some detail, but a brief summary is given here. Vertical shortening can be accommodated relatively easily by the use of *telescopic guides*, and appropriate ‘expansion’ joints on all shaft services. Horizontal dislocations are more problematic: the shaft steelwork and services need to be freed from the shaft sidewall over a distance of about 50 m above and below the reef intersection, and the shaft needs to be excavated with adequate horizontal clearances to accommodate the displacements involved. Tensile induced strains, which can potentially damage the shaft lining and steelwork, will also feature both above and below the zone of freed steelwork.

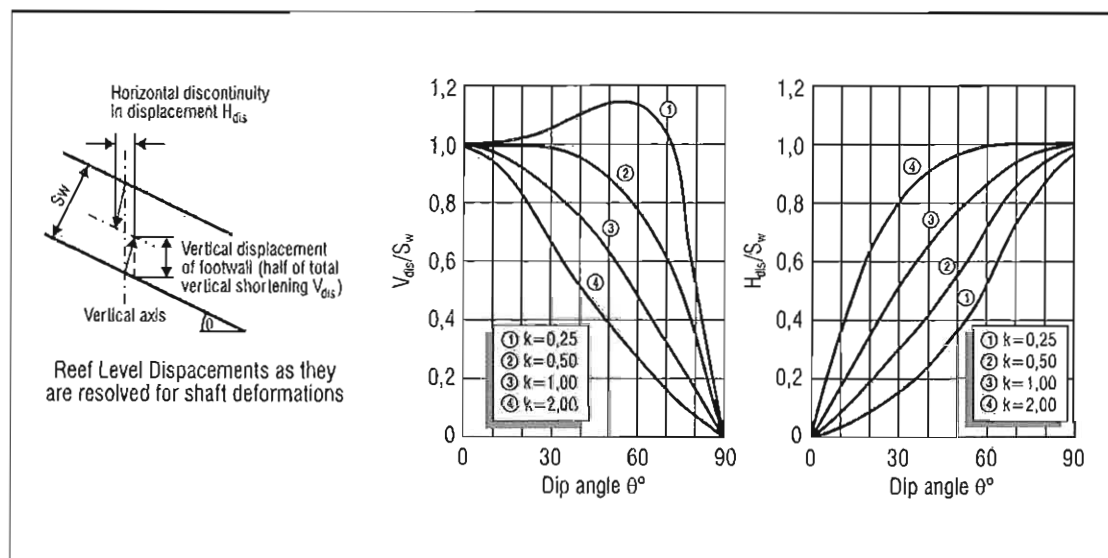


Figure 7.3.6 Expected maximum reef-level dislocations on removal of shaft-reef (k is the horizontal:vertical virgin stress ratio)

The effective magnitude of these dislocations can be greatly reduced, or even eliminated, by one or more of the following means:

- i) **Delayed installation** of permanent shaft steelwork until the major part (100-200 m radius) of the shaft-reef has been pre-extracted and only small residual deformations need to be accommodated. This mining can most easily be accomplished if (as is sometimes the case) access can be developed from a neighbouring existing shaft.
- ii) The use of substantial regional support (good quality **backfill / crush pillars**) in the inner reef area, but not closer than about 20 m from the shaft to avoid stress-regeneration damage to the brows. This will substantially reduce the effective stoping width (S_w in Figure 7.3.6) and restore compression into the immediate hangingwall, thus eliminating the risk of ground water incursion and also permanently reducing the magnitude of shaft dislocations and lining-damaging deformations.
- iii) In deep environments, the use of '**satellite pillars**' (substantial blocks of unmined ground, or even conventional stabilizing pillars, located a suitable distance from the shaft). Satellite pillars can considerably reduce long-term deformations in the shaft while still enabling overstoping protection at the reef intersection itself, but can lead to seismic or compressive strain problems if positioned too close to the shaft or other important excavations.

These options are extremely powerful, but demand the use of careful numerical modelling in order to quantify as accurately as possible the likely deformations to be accommodated by the shaft steelwork designs, as well as to clarify the other important mining tradeoffs involved.

7.3.6 Shaft Pillar Extraction

The extraction of a shaft pillar late in the life of a shaft demands the most stringent planning, since stress and safety conditions in the pillar are now at their worst. The exercise

can in fact be thought of as the removal of a large and probably seismically-hazardous remnant, containing important installations whose well-being usually needs to be preserved. A **project team**, comprising senior mining, engineering, rock engineering, ventilation and safety personnel, needs to be appointed. This team should produce a report detailing the phasing and scheduling of each aspect of the entire operation. Such a report greatly assists continuity, since the extraction of a large pillar usually takes 2 to 5 years to complete and there are often staff changes during this long a period.

The rock engineering design needs to stipulate the whole stoping sequence from inner pillar stoping to final remnant removal, bearing in mind all geological structures present. In general two stages are involved. The first stage comprises excavation of the **inner pillar** and overstoping protection of service excavations and important haulages / crosscuts. Backfill and/or crush pillars are desirable regional support elements in this zone, except in the immediate vicinity (< 20 m) of the shaft where softer yielding support is indicated. The **outer pillar** can then be removed taking into account constraints of geological structures and any final remnant locations, according to the principles of Chapter 3.5.1. The rock engineering design needs to stipulate also the type, spacing and loading performance of each type of required support; together with the stoping width and regional support specifications to be maintained in the inner pillar area.

It is important that the engineers designing the shaft steelwork modifications be given a reasonable estimate of the vector values of the anticipated displacements (especially the shaft **dislocations**) at all stages of extraction. A first estimate of these values can be obtained from Figure 7.3.6, but for more site-specific and detailed values, numerical modelling is required. A reasonable idea of the in situ elastic modulus needs to be available (values ranging from as little as 30% to 80% of the typical laboratory number of 70 GPa have been found to apply). As in shaft pillar design, the modelled stoping area needs to span the major region influencing the shaft (typically 2 km or more). When making estimates of the displacements, the entire cycle to which the steelwork is subject needs to be considered, i.e. the slow compression of the shaft barrel during the establishment of the pillar, to the decompression once the pillar is removed. It has been established empirically that conventional monolithic concrete lined shafts equipped with steel buntons and 'top hat' section guides can accommodate an induced compressive strain of about 0.4 mm/m, followed by a vertical tensile strain of about 0.4 mm/m, before onset of first damage.

The planning and execution of this phase (**modifications to shaft steelwork, services and lining**) can take up to a year and should be completed before stoping commences. Where anticipated movements are relatively small, slotting of the guides and chairplates or cleats can suffice. This arrangement may require daily attention, and winding speeds may need to be reduced. Where displacements are large, it is advisable to free the buntons from the sidewall in the vicinity of the reef horizon where the calculated strains exceed the tolerable amounts. This can be done by suspending a framework of inter-connected buntion sets with telescopic guides at the bottom and suspending individual sets, or by using a tower of buntion sets supported on a bearer with telescopic guides at the top. In all cases, adequate accommodation for the ride between hangingwall and footwall points must be made.

Pipework and cabling, supported on bearer sets, should traverse as great a distance

either side of the reef intersection as possible. By doing this, the decompression zones in the rock above and below the reef cancel out some of the stope convergence and minimize the number and cost of the compression joints required. **Drainwater** should be diverted around the reef horizon through hoses or open-ended pipes.

The **shaft lining** needs to be secured, as a precaution against slabs falling down the shaft, in the zone around the reef horizon where the vertical stress becomes tensile. In this area it is normal to cover the lining with diamond mesh hung in vertical strips to facilitate taking up the slack at reef horizon when convergence occurs. The mesh is secured to the lining by grouted rebar dowels or rock bolts fitted with flat washers. Lacing is used occasionally but is difficult to keep against the lining because of curvature.

Where it is necessary to keep a shaft operational during and possibly after the shaft pillar is removed, the first objective is to stop the **inner pillar** and secure the shaft/reef intersection. The dimensions of the inner pillar seldom extend beyond 30 m from the shaft. This is usually adequate to decompress the shaft barrel, and it is important that this operation does not lead to a brief but damaging increase in the field stresses around the main chambers associated with the shaft. Where the innermost part of the pillar had not been pre-removed during sinking, it is usually convenient to do so from development ends located within 5 to 10 m of the shaft. These are then stoped towards the shaft from one or two sides on relatively narrow fronts to limit the damage in the shaft. Blast holes should not exceed 70 cm in length, and light charges should be used. When the back of the lining is exposed, it is removed either by a feather and wedge or very light blasts. Mesh secured to the shaft side-walls prevents pieces of lining and rock falling into the shaft. Stopping then proceeds around the shaft until the entire reef and lining is removed. During this operation, the permanent support packs for the inner pillar need to be installed.

After the shaft is secured, the program of stoping of the **outer pillar** should aim to secure the major chambers and reef accessways. Here the structural geology of the pillar often has considerable influence on the configuration adopted to minimize rockburst damage. Full remnant precautions need to be in force at this time.

The **support** chosen depends largely on what the designers wish to achieve and what support has been used in the rest of the mine. Basically the aim should be to avoid concentration of load on the shaft and its chambers while limiting the vertical displacements to those which can be accommodated by the steelwork. In making this selection, the geology of the region surrounding the shaft must be taken into account. There are certain general guidelines which should be used:

- i) At the shaft brow, an area support exerting very little load on the fracture zone should be deployed. This support should be as close to the edge of the shaft as possible. Particular attention must be paid to avoid mobilizing wedges, especially in unlined rectangular shafts.
- ii) Beyond the shaft fracture zone, strong stiff preloaded pack support is usually employed. The idea of this support is to inhibit the development of strata separation in the inner pillar hangingwall. Here too, crush pillars have been successfully used to supplement the packs early in the loading cycle. It should be noted that pack support in the inner pillar area needs to be long life and decay resistant.
- iii) Beyond the inner pillar area, crush pillars, stabilizing pillars, wastewalls or stiff back-

filling or a combination of these should be used to restrict the closure to that which the shaft steelwork has been designed to accommodate. These stiff support types must, however, be carefully located so as to avoid stress reloading of important off-reef excavations.

It is advisable to **monitor** and record the movement on pipe expansion/compression joints and telescopic guides, as well as the stope convergence in the inner pillar and the vertical decompression zones (tensile strains) spanning at least 100 m above and below reef intersection. The daily measurement of guide gauge prior to winding operations is a necessary safety precaution. This practice should continue until the stope convergence in the inner pillar has tailed off. The measurement of horizontal squeeze across the shaft (particularly an unlined rectangular shaft) can be very informative when used in conjunction with the vertical measurements to identify incipient wedge failure.

Daily visual and instrument monitoring of any fault planes intersecting the shaft is recommended to identify the onset of slip. The installation of a device to stop winding operations, in the event of a large seismic event close to the shaft, is strongly recommended where known seismically-active faults intersect the shaft.

The following **case studies** of unexpected failures during shaft pillar extraction are of interest:

1. Wedge failure in a vertical rectangular timbered shaft. In this type of shaft, the dividers and their blocking provide support for the shaft walls; but in this case the blocking of several sets had been removed to enable timbers to be re-aligned to compensate for the dip component of ride that had occurred at reef horizon. Some time after the vertical re-alignment had been completed the vertical displacements measurements in the shaft below the reef showed a downwards movement instead of the expected upwards movement. Examination of the shaft showed a huge rock wedge, defined by two joint sets and the reef plane, which had been driven downwards into the shaft by the skeleton packs on the shaft edge after 40% closure in the inner pillar. The removal of the blocking had permitted this wedge to mobilize. Fortunately, some blocking remained which prevented a serious accident. The wedge was pinned to the rock beyond the joint planes by cable anchors, drilled 2-5° downwards from the shaft. Grouting of these anchors was eventually achieved by plugging the planes with cement pockets and using a thickened grout.

2. Collapse of the shaft fracture zone during shaft pillar removal. The 6-compartment rectangular shaft in question was required to remain functional above 6 level (the uppermost level) to facilitate pumping. Reef intersection occurred between 7 and 8 level. It was planned to operate the shaft to 7 level, the upper limit of the shaft pillar, whilst removing the shaft pillar from another shaft. No attempt was made to install additional support in the shaft as the shaft timbering was intact, access was difficult and costly, and the whole operation was economically marginal. As the shaft below 7 level had been abandoned, no inner pillar was mined. Stopping simply proceeded on breast from one side of the shaft pillar; but when the face reached the shaft position, excessive deformation on 7 level station gave rise to grave concern. It was then decided to try to stabilize the shaft from just below 7 level by doweling the walls. Before this could be achieved, the timber sets from 25 m below 6 level station to the shaft bottom on 10 level collapsed, followed by the appearance of a 3-5 m sidewall fracture zone right round the shaft. This incident graphically illustrated the severity of the stresses involved in the extraction of a shaft pillar, and the absolute necessity for prior removal of the inner pillar in similar operations in order to safeguard the integrity of the shaft itself.

8.1 INTRODUCTION

The deep gold mines in South Africa suffer the highest incidence of rockbursting experienced anywhere in the world. In recent years, the deeper platinum mines have also started registering seismicity and even a few rockbursts have been reported. In this chapter, rockbursting is discussed from an observational as well as a phenomenological point of view, and aspects of rockburst prevention and control are also dealt with.

8.2 ROCKBURSTS - DEFINITIONS

Rockbursts are commonly and pragmatically defined as '*seismic events that cause damage*'. This definition may be inadequate, as it fails to cover the whole spectrum of phenomena that cause rockbursts, nor does it distinguish between the mechanism that causes a seismic event (source mechanism) and that which causes the rockburst damage (damage mechanism) - often these are totally different phenomena. In this book, the definitions developed by the Rockburst Commission Working Group of the International Society of Rock Mechanics (ISRM) will be used.

A seismic event is a transient earth motion caused by a sudden failure of the earth's crust. The resulting emission and radiation of kinetic energy in the form of ground vibrations causes a sensible 'shock' or tremor. Depending on many factors, this energy may or may not result in damage to underground or surface structures. The term is applied to events ranging from the largest earthquake to the smallest of events which result in a barely-detectable tremor close to the source - a range of 10 orders of magnitude. The magnitude of vibration at the source is commonly described in terms of Richter magnitude.

A rockburst is the sudden and violent disruption of rock or disturbances of excavation walls in mines, which is caused by, or accompanied by, a 'shock' or tremor (seismic event) of sufficient magnitude to cause obvious damage to excavations and support, or widespread simultaneous falls of rock. A rockburst is a consequence of mining activity.

Injury to persons (or other damage) resulting from an isolated fall of rock does not necessarily indicate the occurrence of a rockburst, although a tremor from a distant event can cause the fall of an unstable piece of rock. Such an occurrence might be designated a 'fall of ground associated with a seismic event'.

Types of rockbursts. The nature and intensity of rockburst damage due to a single seismic event may vary widely, and several terms may be used to describe the phenomena associated with a single event. The terms discussed below, which are in common use to describe rockbursts, are based either on the **source** or **damage** mechanism.

8.2.1 Rockbursts Classified In Terms Of The Source Mechanism

i) *Strain bursts* are triggered by small changes in the stress field which cause the strength of highly stressed rock to be exceeded, and strain energy stored in the rock mass to be released. [The explosive failure of a hard rock specimen in a 'soft' testing machine forms a very close analogy of the mechanisms involved.] The triggering agent may be a transient stress change associated with a (distant) seismic event, a stress change brought about by the advance of a nearby face, or the result of a time-dependent stress redistribution; but this causative activity only accounts for a small fraction of the energy associated with the rockburst. In this type of rockburst the association between the rockburst and seismic event is direct - the mechanism of the seismic event is the rockburst, and vice versa.

Strain bursts are characterized by the violent failure of intact wall rock of excavations. The effect is usually localised to an area of less than a few square metres and occurs in the immediate wall rock. On occasions, up to a few cubic metres of intact rock located outside the fracture zone of an excavation may become over-stressed and fail violently, ejecting the previously fractured rock between the excavation and the newly failing rock. Strain bursts associated with stress redistribution usually occur in the vicinity of newly exposed rock surfaces.

ii) *Face-parallel bursts* mainly affect the rock in the immediate vicinity of stope faces. The source mechanism of the seismic event associated with this type of rockburst is usually a shear rupture ahead of the stope face, and probably initiates in intact rock beyond the periphery of the fractured ground surrounding the stope. Such a shear fracture in the hangingwall typically propagates downwards in *en echelon* segments towards the stope, dips towards the back area and has a normal sense of displacement. This displacement causes wedging and crushing of the fractured rock ahead of the face, stress redistributions which may cause local strain bursts, and on occasions secondary low-angle fractures are formed which dip up and away from the face. Similar ruptures and deformations, in a mirror-image sense, occur in the footwall.

The violent compression of already fractured rock results in rapid dilation of the rock ahead of the face causing ejection or buckling of slabs on the face, violent peeling off of rock from new fracture surfaces in the hangingwall and severe shake down of fractured rock from the hangingwall due to the proximity of the seismic event, which can be of magnitude up to about $M = 3$ but usually less. Footwall heave into the stope is often experienced due to shearing on bedding planes or pre-existing fractures in the footwall, causing buckling of the immediate footwall beds.

iii) *Pillar or remnant bursts.* Pillars or remnants attract load caused by mining in their vicinity. If such blocks of ground are of critical dimensions, they can become over-stressed and fail violently, either totally or in part. The damage occurs peripheral to the pillar; and dynamic closure of 100 mm or more can be expected in the immediate vicinity, the closure dying away concentrically from the focus area.

Similar to the case with strain bursts, the failure of the pillar is the rockburst and causes a seismic event of magnitude ranging from about $M = 0.5$ to $M = 2.5$.

The underlying cause of the failure is transfer of induced stress to the core of the pillar which becomes over-stressed and fails. The final additional stress which overloads the pillar can be due to time-dependent fracturing of the edges of the pillar transferring load to the core, or due to seismic stress waves from a distant event impinging on a pillar in a state of unstable equilibrium.

iv) *Pillar foundation failures.* In deep longwall mines, a system of systematic strike-aligned stabilizing pillars has been implemented for regional support. Rockbursts have occurred where the seismic events locate beneath the pillar. This has been interpreted to indicate the 'punching' of the pillar into softer footwall strata, creating new shear ruptures, and resulting in stope closure and shakedown damage. The seismic events associated with pillar foundation failures do not generally exceed $M = 3.5$.

v) *Slip on geological structures.* Geological structures such as faults and dykes represent zones of potential weakness in the rock mass. The redistribution of stress associated with mining may trigger slip along one of these pre-existing structures. As these features may extend for hundreds of metres or even kilometres, a large volume of stressed rock may be involved and the magnitude of the event can be very large (up to $M = 5$). Any excavations in the near-field may suffer severe damage, while shakedown may affect stopes and tunnels over a large area; strain bursts may also be triggered.

A useful 'rule of thumb' relating the magnitude M and source dimension L_s (the linear distance over which slip occurs) of a seismic event is:

$$\log_{10} L_s \text{ (in metres)} \cong 1 + M/2$$

$$\text{or } L_s \text{ (in metres)} \cong 10^{(1+M/2)}$$

For example, for $M = 2$, $L_s = 100\text{m}$; for $M = 4$, $L_s = 1000\text{m}$; for $M = 5$, $L_s = 3000\text{m}$; approximately.

Any point located at a distance less than $1/2 L_s$ from the seismic source is said to be in the **near-field**; sites further away are in the **far-field**.

Near-field deformations decay rapidly with distance from the source, and thus the rockburst potential in the near-field is highest near the source rupture plane. A preliminary estimate of the width, normal to the rupture plane, of this zone of maximal rockburst damage potential is about $1/10 L_s$. The size and shape of this potentially damaging "near-field" zone, in which ground velocities exceeding 1m/s are likely, is probably dependent on the magnitude and nature of the stress-drop on the rupturing plane, and on the radiation pattern from the source.

8.2.2 Rockbursts Classified In Terms Of The Damage Mechanism

It is important to note that the ground motion on the 'skin' of an excavation may be amplified relative to the ground motion in solid rock a similar distance from the seismic source, owing to the free-surface effect, surface waves and local site effects. In addition, the duration of shaking on the skin of an excavation may be much longer than experienced in solid rock, owing to the trapping of energy in the fractured rock surrounding a deep excavation and the creation of surface waves. This may lead to an accumulation of slip along fractures, the shakedown of blocks, and the unravelling and collapse of the rock wall.

I. '*Near-Field*' rockbursts are the most serious and severe rockbursts. They are associated with medium to large seismic events resulting from either shear of intact rock or slip on a geological structure. The damaged excavations are in the '*Near-Field*' of the seismic event and are affected by the dynamic deformations of the rock mass in this region. Peak strong ground motions in excess of 3 m/s may be experienced in this volume of rock.

Damage caused by such rockbursts is severe and sometimes catastrophic, with large sections of stopes and tunnels having to be re-established or abandoned. A characteristic of this type of rockburst is significant co-seismic closure of excavations, occasionally resulting in total closure. Support units are frequently damaged; steel props may be punched into the wall rock; packs, particularly on gullies, are often broken and displaced; tendons in gullies and tunnels are either ineffective, rock between them being ejected, or are snapped or sheared; fabric support is often over-extended and broken, spilling rock into excavations; shotcrete is broken and rendered ineffective as a support; the periphery of backfill will show signs of distortion, and fill material may be expelled into the face area and gullies. Extensive falls of ground and collapse of excavations occurs. Much of the ejected rock is in smallish fragments, indicating co-seismic fracturing.

Displacements of the order of cm or tens of cm can be observed on the sheared geological structure where it intersects excavations. Away from the source region, rockbursts grade into 'violent shakedown bursts' and 'shakedown bursts' as the intensity of shaking and triggering of strain bursts diminishes.

II. *Violent shakedown bursts* experience somewhat lower ground velocities than near-field bursts. They involve severe and prolonged shaking of the whole stope or tunnel. Rock blocks (perfectly stable under gravitational loading) are shaken loose - key blocks are ejected and unravelling occurs. Dynamic closures of 10 - 100 mm may occur, and as a result some ejection from faces may be evident. Damage to stope support is limited - temporary non-yielding support units may be buckled or broken and some headboards may be damaged. Limited footwall heave can be experienced. Inadequately supported gullies are often seriously affected.

In tunnels, rock between tendons can be dislodged and bulking of fractured rock causes bulging of fabric support. Poorly installed or deteriorated support elements may fail. Sections of footwall heave may occur. Cracking and minor failure of shotcrete can be observed. Ventilation doors can be damaged and trains of major equipment can be misaligned. Where very blocky or otherwise exceptionally poor ground conditions existed, total failure of the support system can occur.

The seismic events associated with these rockbursts are usually large ($M > 2$) but are located sufficiently far away that the damage is in the far field of the event and peak ground velocities can be estimated, for example from Figure 1.4.8 in Chapter 1.4.

III. *Shakedown bursts*. Loose rock can be shaken down, subjected to either a short sudden jolt from transient seismic waves from a nearby small seismic event, or to lower frequency waves from more distant, larger events. There is generally no damage to support or dynamic closure of excavations. The mechanism is not very violent and support systems designed to prevent gravity-induced rock falls should also prevent this type of rockburst damage. Shakedown bursts grade into damage that may be described as a 'fall of ground associated with a seismic event'.

8.3 DISTRIBUTION AND INTENSITY OF ROCKBURST DAMAGE

The severity of rockburst damage often varies greatly. One panel in a longwall may be severely damaged, while an adjacent panel (perhaps even closer to the focus of the seismic event) is unscathed. The condition of a tunnel may change from being sound to a state of total collapse over a distance of a few metres. Why is this so? A series of rockburst investigations were carried out under the auspices of SIMRAC, as it was believed that a detailed understanding of both the source and damage mechanisms, and the application of this knowledge to the design and support of excavations, could lead to a reduction in the hazard posed by rockbursts. In this section, four rockburst investigations are presented in detail, followed by a synthesis of the findings of the series of over 30 studies.

8.3.1 Rockburst Investigation Methodology

The following procedure is currently adopted in the investigation of a rockburst:

1. A team of specialists visits the site shortly after the event, in most cases prior to any rehabilitation so that the only disturbances have been due to rescue operations. Each member of the team has a good general knowledge of rock engineering and mining, in addition to an area of specialist expertise (e.g. layouts, support, geology, seismology). The damage to the excavation and support elements is carefully studied, dynamic closure is estimated, and mining-induced fractures, joints and other geological features are recorded. Interviews are held with witnesses to the rockburst, and rock engineering staff at the mine.
2. Seismograms of the incident are used to determine the seismic source parameters.
3. The seismic history of the area in the vicinity of the rockburst and of nearby structures (dykes and faults) is assessed.
4. Numerical modelling is used to evaluate the mining layout and sequence at the time of the rockburst, by calculating parameters such as Energy Release Rate and Excess Shear Stress.
5. Support elements such as props and tendons may be recovered from the rockburst site, and tested in the laboratory.
6. Rock samples may be collected, so that the properties of the strata can be determined.
7. Future mining strategies are investigated, and recommendations formulated.

8.3.2 Case History 1: Pillar Burst Triggering Face Burst And Violent Shakedown

A seismic event with local magnitude $M = 2.1$ caused a fatal injury and significant damage to a working place 2300 m below surface in a mine in the Carletonville area. A peninsular remnant of VCR was being mined at the time. Figure 8.3.1 is a location plan showing the site of the rockburst. A cross-section through the rockburst site parallel to strike is shown in Figure 8.3.2.

Observations at the rockburst site. The mining-induced extension fracturing exhibits features typical of VCR areas where quartzite/conglomerate comprises the

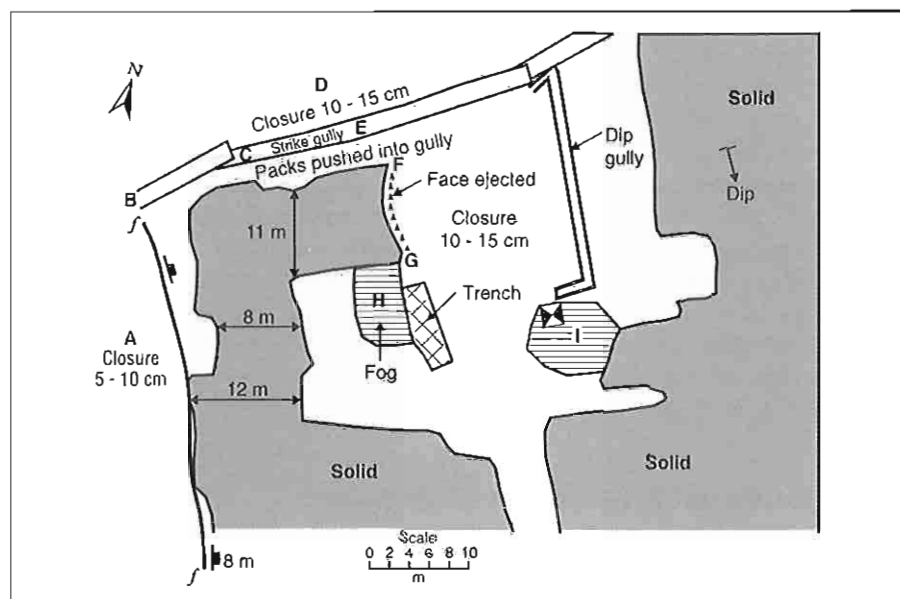


Figure 8.3.1 Plan of the peninsular remnant showing the positions of observations made during the inspection of the rockburst site

footwall and hard lava comprises the hangingwall. Flat fracturing occurs within the hangingwall lavas, while steep fractures are encountered in the quartzite footwall.

A normal fault with an 8 m up-throw forms the western boundary of the remnant. Inspection of the fault plane, where it had been exposed by mining, showed no evidence of recent movement. Fresh timber was exposed in a gully pack at point A, indicating that dynamic loading had occurred, with an estimated dynamic closure of 5-10 cm. The area around the upper strike gully was inspected. Some of the down-dip siding packs had been pushed into the gully between C and E by broken rock ejected from the exposed down-dip face. Fresh splits in the timber indicated that dynamic closure on packs had occurred between C, D and E, where dynamic closure was estimated to be 10-15 cm. The breast face had been stopped along the line F-G and a line of end-grain packs installed, as the reef rolled down towards the west by about 2 m. Updip mining had then been commenced to extract this block of ground, with trenching required to establish the updip face. The brow to the west of the roll (H on Figure 8.3.1) collapsed during the rockburst up to a bedding plane fault, which to the east of the roll formed the hangingwall of the stope, but to the west continued into the lava 1.5 m in the hangingwall (Figures 8.3.2 and 3). A large amount of rock was also shaken from the roof at position I, burying a winch which fortunately was not being operated at the time of the rockburst.

A small amount of intact reef could be seen in the stope face at the top of the breast panel (position F). The hangingwall lava was exposed over the remainder of the face. Between F and G, rock had been ejected into the void between the original face position and the first dip line of packs, typical of a face burst (Figure 8.3.4). It was clear that no hangingwall had fallen. Fresh splitting on packs in this area also indicated dynamic closure of 10-15 cm.

Assessment of support performance. The support system in the working place consisted of hydraulic props with headboards at the face, and 1.1 m x 1.1 m timber packs. Although many of the hydraulic props had been removed during the rescue effort, it was clear from examination of the remaining props that they had been ineffective in preventing falls of ground associated with fragmentation of the hanging-wall. The pack support underneath the brow at H had been well installed, but clearly had not been adequate to hold the brow up. No horizontal confinement was provided, without which the brow was prone to disintegration when subjected to violent shaking. The shepherds crooks installed in the vicinity of the crosscut-reef intersection (position I) were also ineffective in preventing significant shake-out of the fractured hangingwall.

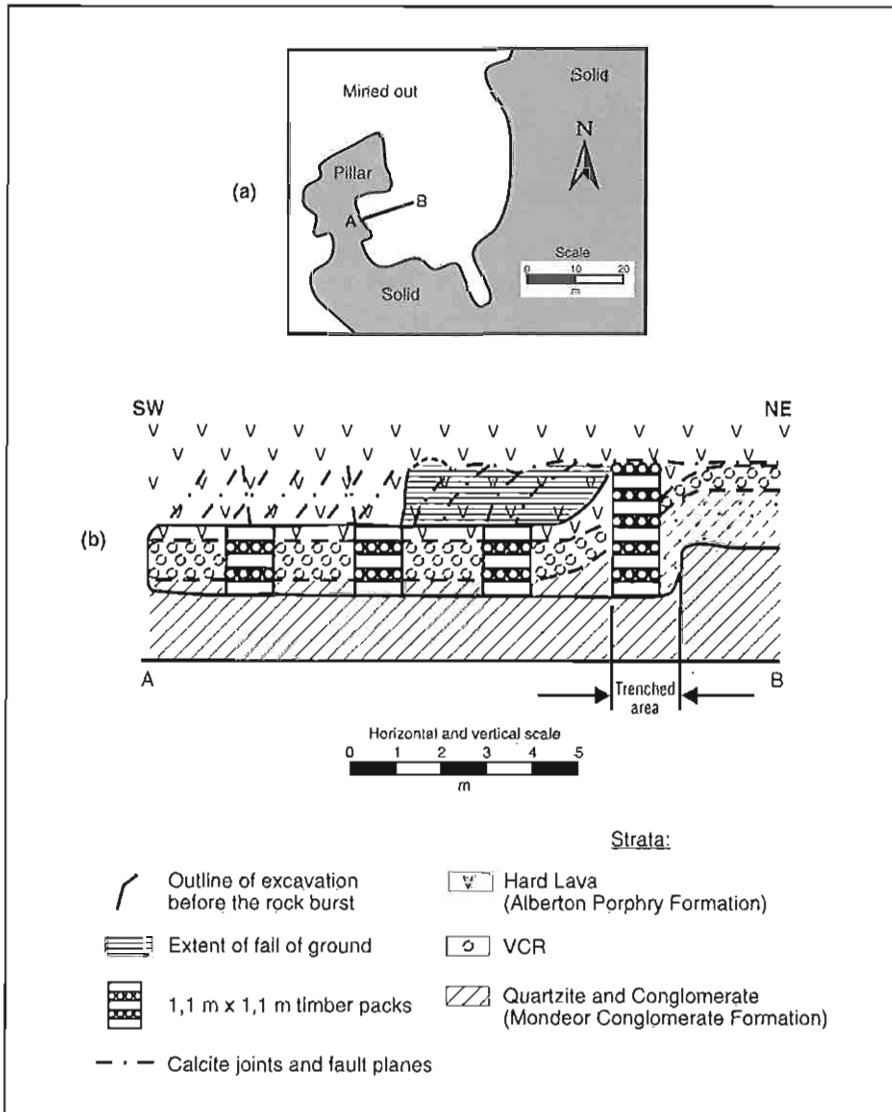


Figure 8.3.2 Cross-section through the updip working place



Figure 8.3.3 Photo-mosaic showing the face, viewed from position H (Figure 8.3.1) and looking updip. The timber packs on the right are situated to the east of the roll. The corresponding section is shown in Figure 8.3.2. The hangingwall fragmented during the rockburst. Most of the fallen rock was subsequently removed. Note the numerous fractures and joints in the face, and the smooth surface marking the parting above the collapsed brow.



Figure 8.3.4 Rock ejected from the footwall and face between F and G (see Figure 8.3.1) into the space between the original face position and the first dip line of packs. The photograph is taken looking updip with the face to the left. Note the intact hangingwall.

Assessment of layout. Numerical modelling was conducted to determine whether or not a high seismic hazard was indicated. The main results of MINSIM modelling are summarized below.

- The energy release rate (ERR) on the advancing faces was low ($< 20 \text{ MJ/m}^2$) on account of the limited mining span, giving no indication of potential face bursting.
- The major principal stress on a dyke about 50 m south-east of the remnant was low (85-95 MPa), and the changes in the stress due to mining were small (less than 2 MPa). No slip on the dyke was indicated.
- The average pillar stress (APS) on the pillar was estimated at about 190 MPa, and taking a value of 180 MPa to be the minimum UCS value of the host rock, the APS:UCS ratio was less than 1.1. This is safely below the limit of 2.5, suggested in Chapter 3.2.8, for avoiding pillar foundation failure. However, the APS value was derived from elastic analysis, which does not take into consideration the fracturing of the faces of the remnant. In reality these areas would be crushed and the load transferred to the interior of the remnant. This would reduce the effective width of the pillar from 9-13 m to, say, 5-9 m and would almost double the APS on the remnant to 350-400 MPa, a level where failure (on an effective 4:1 width:height ratio pillar) becomes much more likely.

Rockburst mechanism. The focus of the rockburst was close to the boundary of the mine and outside the mine seismic network, resulting in poor location accuracy. Consequently it was not possible to determine from the seismograms alone whether the focus of the rockburst was associated with slip along a nearby dyke or failure of the peninsular remnant. Calculations were made to determine whether it was feasible for the failure of the remnant to release sufficient energy to produce a magnitude $M = 2.1$ event. The greatest uncertainty in making this calculation is the estimate of the volume of closure; it was nevertheless possible to get a fairly reliable estimate of local closure magnitudes from observations of the damage to packs surrounding the remnant. It was also necessary to estimate the area of the stoping which sustained this closure due to collapse of the remnant. The following parameters were used in the calculation:

Equiv. area of stoping sustaining closure	1000 m ²
Max. closure	100 mm
ΔV_c (volume of closure)	100 m ³
G (rigidity modulus of quartzite)	$4 \times 10^4 \text{ MPa}$

The seismic moment can be estimated using an empirical relationship of McGarr & Wiebols:

$$M_0 = G \Delta V_c = 4 \times 10^{12} \text{ Nm}$$

The magnitude of the event can then be estimated using the Hanks-Kanamori moment magnitude relationship:

$$\log_{10} M_0 = 1.5 M + 9.1$$

and $M = (12.6 - 9.1)/1.5 = 2.3$.

Thus it is entirely possible that the $M = 2.1$ event was caused by pillar failure. On the basis of the above calculations and the distribution of damage, it was concluded that failure of the peninsular remnant was the source mechanism of the rockburst, rather than slip along the dyke.

Conclusions

1. The source mechanism of the $M = 2.1$ rockburst is thought to be the failure of the peninsular remnant, and not slip on the nearby dyke or fault.
2. The peninsular remnant was particularly vulnerable to rockbursting due to:

- a. Its L-shaped geometry.
 - b. A high effective unconfined pillar height due to trenching and bedding-parallel faulting.
 - c. A weak hangingwall due to faults and joints in the vicinity of the roll in the reef.
 - d. The rock type exposed by stoping around the peninsular remnant is mostly the hangingwall lava. While the hard Alberton lava hangingwall and the VCR have similar values of Young's modulus (about 80 GPa), the UCS of the lava (about 250 MPa) is substantially greater than that of the VCR (about 180 MPa), and consequently can store more elastic energy. Should a lava remnant fail, it will do so more violently than the case where the remnant is comprised of conglomerate.
3. Elastic modelling did not indicate any significant transfer of stress onto the nearby dyke or indicate immediate failure of the remnant. However, if the reduction in the effective width of the remnant due to fracturing of the face is taken into account, the average stress (APS) in the remnant increases to 350-400 MPa, and failure is likely at these stress levels.
 4. The stope support system (packs and hydraulic props) was ineffective in preventing falls of ground due to fragmentation of the hangingwall. The lack of support providing horizontal confinement to the brow probably increased its susceptibility to fragmentation. Shepherd's crooks were ineffective in preventing shake out of the hangingwall in the vicinity of the crosscut-reef intersection.

Recommendations

1. Avoid the formation of rectangular or L-shaped peninsular remnants. Rather, in this example, consider carrying out underhand mining to safely reduce the volume of the remnant whilst providing a stable lower section. This is a small-scale example of the principle of 'mining towards the solid'.
2. If trenching is carried out, install support that will provide lateral confinement to the brow.
3. Special care should be taken to support the hangingwall when mining in the vicinity of rolls, especially as the frequency of weak calcite-coated joints appears to increase in these areas.
4. Always equip hydraulic props with headboards in areas where the hangingwall is fragmented, to improve the effectiveness in containing falls of ground.
5. The interpretation of calculations of stress and ERR must be done with extreme care. Simplistic elastic modelling techniques do not take the fracturing of the face into account, and can produce unrealistically high values of stress at the edges of pillars and abutments. In reality these areas fracture and crush, shifting the load away from the face. Thus, the core of the pillar is subjected to greater stress than is indicated by the numerical model, and this can become critical when the pillar dimensions are small – particularly when (as in this case history) the effective height of the pillar is increased by trenching.

8.3.3 Case History 2 - Rockburst Damage To Tunnels

An $M = 3.6$ seismic event caused extensive damage to haulages about 2500 m below surface in a mine in the Carletonville district (Figure 8.3.5). The epicentre of the event was located some 400 m south of the damaged tunnels. The area is overlain by the Ventersdorp Contact Reef (VCR), which is enclosed by lavas belonging to the Klipriviersberg Group in the hangingwall and shales, shaley quartzites and quartzites belonging to the Jeppetown Subgroup in the footwall. The seismic event affected

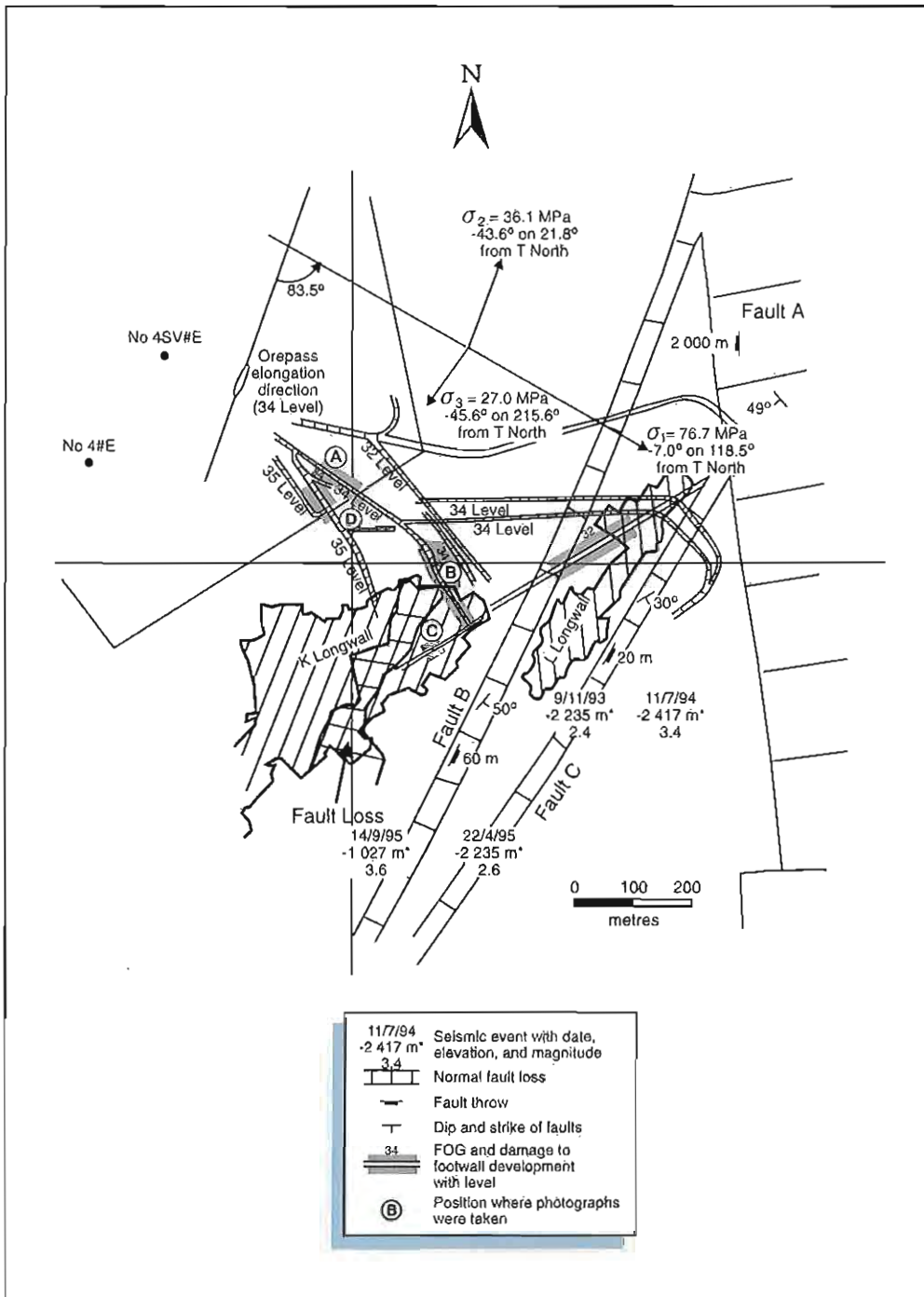


Figure 8.3.5 Plan showing damage to tunnels in a gold mine in the Carletonville area caused by an $M = 3.6$ seismic event on 14 September 1995. The locations of all seismic events with $M > 2$ which occurred in the preceding two year period are also shown. Falls of ground (FOG) and damaged sections of tunnels are indicated by shading. In situ field stress measurements are also indicated.

excavations from 31 to 36 levels, a vertical distance of about 200 m, over a wide area. The damage was mainly in crosscuts and drives that provide access to two longwalls (which were virtually unscathed) situated to the south-east of the shaft. This area has a history of large seismic events, many of which have caused damage to the access development, with stopes being less severely affected. This strongly suggests that the seismic events are related to geological structures remote from the stopes, and possibly also to abnormal virgin stress fields in the area.

Observations at the rockburst site

The damage on 34 and 35 Levels was generally associated with the north-eastern sidewall and the hangingwall of the excavation where the tunnel was oriented in a south-easterly direction, i.e. parallel to the local strike. A key factor controlling the distribution of the damage was the type of support in place at the time of the rockburst. Some sections were supported by grouted rebars, mesh, lacing and shotcrete, and suffered little damage. Other sections, where mesh and lacing had not yet been installed, sustained severe damage. In a tramming loop east of the shaft, the southern sidewall was severely damaged for a short distance where the loop approached the main development (Figure 8.3.6). A section of another footwall drive (site B) was completely blocked by the bulking of the sidewall (Figure 8.3.7). Footwall heave from a north-easterly direction was also evident in the footwall drives in almost all places where damage was noted. In a 34 level cross-cut, damage was confined to a 20-30 m long section where bad rock conditions had been encountered in the past. In this area both sidewalls were damaged, and footwall heave



Figure 8.3.6 Damage to 34K footwall drive at Site A showing falls of ground between grouted shepherd's crook tendons. Mesh and lacing had not yet been installed in this section of the tunnel. In this area, the south-western (right hand) side of the drive was most severely damaged.

appeared to be more prominent on the north-western sidewall of the crosscut. The damage on 35 level was generally less severe than on 34 level. The damage was characterised by the collapse of the northeastern sidewalls and falls of the hangingwall.

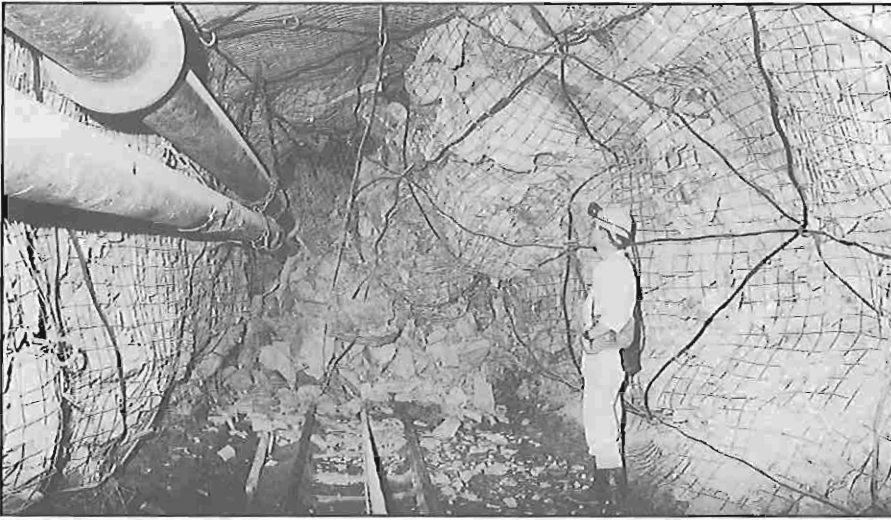


Figure 8.3.7 Damage to 34K footwall drive at Site B showing complete closure of the tunnel.

Assessment of support performance

The influence of support on the distribution of damage was clearly evident. In areas where grouted rebars were the only means of support, damage was very severe. In areas where wire mesh and lacing was installed, sidewall and hangingwall damage was evident to varying degrees. In some places serious bulking of the northeastern sidewall and hangingwall made bleeding and resupport of the haulages necessary for re-opening. In areas where shotcrete had been applied there was no damage at all (Figure 8.3.8).

Failure of the support system was due to either direct tensile or shear failure of the shepherds crook rebar rock bolts. Most of the failure was fresh and was probably associated with the $M = 3.6$ seismic event, however one example of an old failure (rusted failure surface) was exposed by the collapse of the rock mass. Direct shear failure of the bolts was predominantly associated with the hangingwall of excavations at the position of clearly defined bedding planes, where clean guillotining of the bolts had occurred, with the sense of shearing being in the down-dip direction (Figure 8.3.9). Within the sidewall of the excavation, tensile failure of the bolts was generally observed. The majority of the bolts, however, indicated some degree of plastic bending prior to failure, which suggests some component of shear movement.

The most extensive failure of the tunnel excavations occurred in areas that had not been meshed and laced. This is considered to be caused by the inadequacy of the rebar reinforcement alone under rockburst conditions, where the failure of a single support unit would result in damage to the excavation. Within the meshed and laced tunnel sections, the higher integrity of the support system meant that even if individual support units within the system failed, the remaining support units could still pre-

vent collapse. Both shear and tensile modes of failure of the rebar support occurred within the meshed and laced areas of the tunnels, but the system integrity was largely maintained. However, should the excavations be subjected to further seismic loading, total failure of the system could well occur.



Figure 8.3.8 A haulage on 34 level showing the effect of the support system on damage. The section of the tunnel in the foreground was supported only by grouted rebars, and significant damage was sustained. The support system in the section of the tunnel beyond the miner consisted of grouted rebars, mesh, lacing and shotcrete. Damage was negligible.

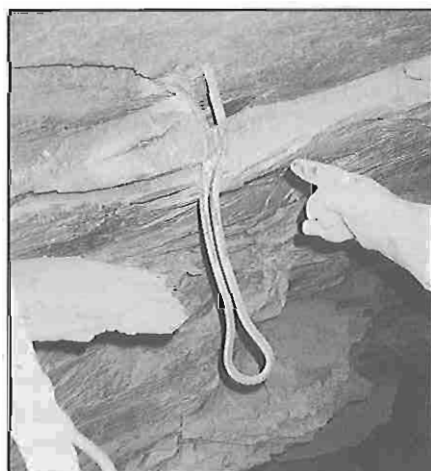


Figure 8.3.9 Kinked shepherd's crook rebar observed in the hangingwall of the 34K footwall drive between Sites B and C. Some bars had been sheared by the movement along bedding planes.

Assessment of layout

The virgin stress tensor is often assumed to comprise a vertical major component

proportional to depth, and two roughly equal horizontal components which at this depth (2500 m) would be about one-half of the vertical stress – Figure 1.3.1a. In short, the ‘expected’ virgin stresses are $q_v \approx 68 \text{ MPa}$, $q_H \approx q_h \approx 34 \text{ MPa}$. Observations of tunnel fracturing, however, indicated that the usual assumption of the stress state at depth did not appear to be valid in this area, especially towards the east. The in situ stress state was measured, sometime prior to the seismic event discussed here, by drilling and overcoring three horizontal boreholes into a tunnel sidewall, and taking strain relief measurements. It was found that the maximum field stress (σ_1) was almost *horizontal*, and orientated northwest to southeast – Figure 8.3.5. The orepass in the 34 Level tip crosscut was also examined, and was observed to have a markedly elliptical cross-section, with the long axis of the ellipse orientated on a bearing of 21° from true north. This suggests a major horizontal stress direction on a bearing of 111° , which is in excellent agreement with the bearing of 118° for the major compressive stress component measured in the tunnel. There is always some uncertainty attached to in situ virgin stress measurements, and in this example, the very low value of vertical component measured (about 32 MPa, less than one-half of the cover load of 68 MPa) casts some doubt on the validity of the measurements. Nevertheless, it is clear that the virgin stresses in this area are *highly abnormal*, due, perhaps, to significant geotectonic ‘hangup’ in fault A which has a throw of some 2000 m.

The MINSIM suite of programs (Chapter 11.4.4) was used to analyse the stresses and displacements induced by mining, in order to throw further light on how such a relatively insignificant amount of mining could generate such a strong seismic response in the area. The elastic constants of the rock-mass were assumed to be 70 GPa and 0.2, for Young’s Modulus and Poisson’s Ratio respectively. The Excess Shear Stresses (ESSs) on selected geological structures were compared using either the measured (k-ratio ≈ 2) or ‘normal’ (k-ratio = 0.5) stress states (Table 8.3.1). The modelled B fault appears unaffected by the different *in-situ* stress states, while the A and C faults are distinctly less stable when compared with the ‘normal’ stress state. These results come from a static elastic model, and as such should not be interpreted as quantitative seismicity predictors. What the model *does* indicate is that there is probably a higher potential for seismicity on some structures (e.g. certain faults and dykes) because of a combination of their orientation and the local stress state. More underground stress measurements, seismic data and numerical-modelling are necessary for a clearer identification of features responsible for seismicity in this area.

Table 8.3.1 Summary of ESS Results (MPa) for Three Benchmark Sheets

Dip	Fault A 49°SW	Fault B 50°SE	Fault C 30°SE
‘Normal’ Stress State (k-ratio = 0.5)	-12.1	-12.5	-22.5
Measured In-Situ Stress State (k-ratio ≈ 2)	-4.5	-12.6	-14.9
Difference between Normal and Measured	+7.6	-0.1	+7.6

Rockburst mechanism

The *peak particle velocity* at one site was estimated from a large block, attached to a failed rebar support, that had been ejected from the hanging wall. The bar protruding from the block exhibited necking for a distance of about 10 mm. It may be assumed that similar necking occurred on the section of the bar still grouted in the roof. It can

further be estimated that the bar was extended by about 10 mm before failure.
Kinetic + Potential energy of block $\geq$ Elastic energy absorbed by bar up to failure

$$E_k + E_p \geq E_f$$

where m = mass of block = 800 kg; h = extension of bar at failure = 0.01 m;

g = gravitational acceleration = 10 m/s²; Tensile strength of tendon = 150 kN

E_f = area under force-displacement curve $\cong 150000 \times 0.01 = 1500$ J

$E_k = 1/2 m v^2 = 400 v^2$; $E_p = m g h = 80$ J

$\therefore v \geq 1.9$ m/s

Conclusions

In summary, the following conclusions were reached :

1. The seismic event appeared to be remote from the mining excavations, and was most likely associated with movement on one or more faults in the area.
2. The anomalous stress state in the region was probably responsible for the instability on the geological features.
3. The damage in the access development was controlled partially by the friability of the rock masses involved, but very largely by the quality of the installed support.
4. The partial failure of the mesh and lace support system was due to the limited yield and shear resistance capacity of the grouted rebars.

Recommendations

The following recommendations were made:

1. Long-term excavations that are expected to experience seismicity, should be supported by pre-stressed yielding units that can accommodate shear deformations (e.g. grouted rope anchors, cone bolts) integrated with mesh and lacing. In areas close to potential seismic sources (faults, dykes, abutments) where severe damage is expected, these support units should be supplemented with shotcrete. In short-term excavations, support tendons with limited yield capacity (e.g. smoothbars) that are cheaper than cone bolts or long rope anchors may be installed, but should still be integrated with wire mesh and lacing. [See Chapter 6.4.]
2. Further in situ stress measurements should be undertaken to determine the area over which the anomalous stress state is found. All available structural, surface reflection seismic and *in situ* stress data should be combined to identify and locate seismically active structures.

8.3.4 Case History 3 - Violent Shakedown Burst Caused By Slip On Dyke

An $M = 2.5$ seismic event caused multiple fatalities and damage in a Carbon Leader Reef (CLR) stope about 3000 m below surface in a mine in the Carletonville area (Figure 8.3.10). The epicentre of the event was located about 50 m ahead of the W2 panel. In this stope, the CLR dips to the south at approximately 20°. The hanging-wall of the CLR is a package comprised, from bottom to top, of a siliceous quartzite horizon about 1.20 m in thickness, the Rice Pebble Marker (a matrix-supported small pebble conglomerate, 20 cm or more in thickness), and the Green Bar (a predominantly shale horizon). The footwall of the CLR is a massive, competent quartzite. A dyke intersects the W3 panel of the longwall; it is 5-15 m in thickness, has a NE-SW strike, and dips to the west at 70°. The CLR is down-thrown by about 15 m to the west of the dyke.

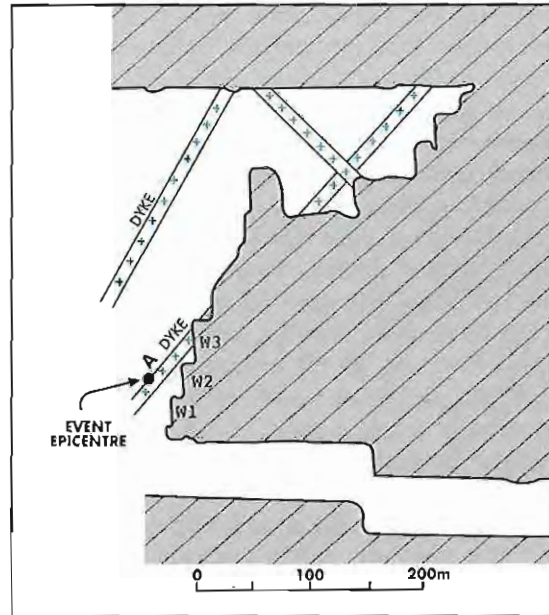


Figure 8.3.10 Plan showing the location of the $M = 2.5$ seismic event which caused a rockburst in the W3 panel.

Observations at the rockburst site

The W3 panel was part of a longwall being mined on breast in a westerly direction (see Figure 8.3.10). The most severe rockburst damage was found in the W3 panel. The W1 and W2 panels, located downdip and closer to the focus of the $M = 2.5$ seismic event, were virtually undamaged. The key question to be answered in this instance is: what was the cause of the intense damage in the W3 panel?

The fall of ground in the W3 panel extended from the downdip strike gully to the intersection with the dyke, a distance of about 20 m (Figure 8.3.11). A height of about 1 m of the hangingwall in this area had collapsed. The distance from the face to backfill was about 3.5 m. The fall of ground was bounded on the updip side by a weak joint, filled by a quartz vein locally containing pyrite; to the west by face-parallel fractures; to the east by the backfill; and to the south-east by face-parallel and longwall-parallel fractures. Part of the hangingwall remained standing in the W3 panel and formed a brow, in which openings of 1 cm or more had formed along bedding planes. No finely crushed rock was observed in these partings, but rather coarse quartzite aggregate a few millimeters in diameter, interpreted to indicate tensional failure. About 0.5 m of material had ejected from the stope face of the W3 panel. The rock forming the face was crushed and had a granular texture, indicating that the face had experienced compression during the rockburst. This contrasted sharply with the fractured (but uncrushed) face in the W2 panel. This was interpreted to indicate that the rock between the face of the W3 panel and the dyke had failed, possibly triggered by slip on the dyke. The W3 panel was inspected again after it had advanced by several metres, and, in contrast to the earlier observation, no unusual crushing of the face was evident.

Several sets of mining-induced fractures were observed in the panel: a NNW-striking set essentially parallel to the stope face, perhaps refracted by the lag between the W2 and W3 panels; a NNE-striking set parallel to the overall orientation of the longwall and the dyke; and a set parallel to a set of NW-trending quartz-filled joints. The dip of the mining-induced extension fractures exhibited a turning point of inflection at the level of the Carbon Leader reef, as is typical for reefs with sedimentary bedded foot- and hangingwalls (e.g. Figure 4.2.5). These fractures intersected the ore body perpendicularly. The orientation of the fractures was measured in the arches between gully packs formed by falls of ground. It was noted that the dip flattened to 40° immediately below the Green Bar, some 1.5 m above the stope. The number of extension fractures increased from fewer than 20 fractures per metre to about 40 fractures per metre within 1 m of the NW/SE trending joints. Extension fracturing within the quartzite underlying the Carbon Leader reef was less frequent than in the hangingwall, averaging about 15 fractures per metre. No evidence of significant co-seismic movement could be detected on the extension fractures.

Shear fractures, with visible shear dislocations, were noted at intervals of about 1 m (c.f. Figure 4.2.3). Movement also occurred along the quartz-filled joints that run subparallel to the extension fractures and are most pronounced in the hangingwall. These joints were encountered in the hangingwall of the areas where falls of ground occurred. The shear plane that runs close to the contact between the dyke and the abutting rocks of the CLR zone also sustained movement.

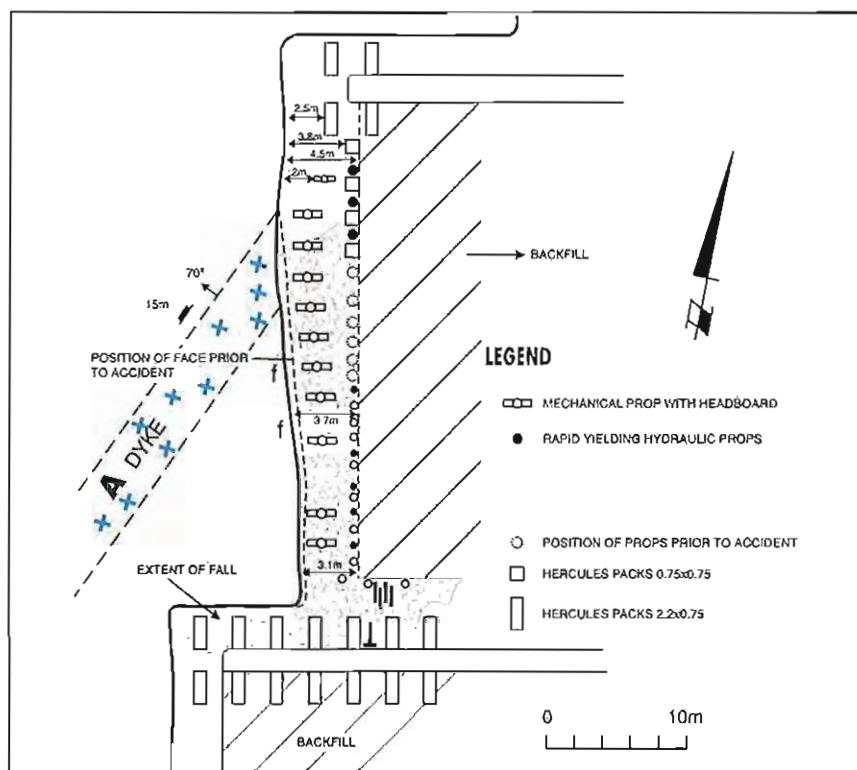


Figure 8.3.11 Plan showing the support installed in the W3 CLR panel, and the extent of the fall of ground during the rockburst.

The contact between the dyke and country rock was irregular, with a planar shear zone positioned along a pseudotachylite intrusive. The contact was inspected again after the face had advanced by several metres. The contact was straight with pseudotachylite present along the interface, but the shear plane was absent, implying that the shear along the dyke/host rock contact was restricted to the vicinity of the face at *the time of the rockburst*.

The hangingwall was intensely fractured along the strike gully. Falls of ground were observed between packs on both sides of the strike gully, forming arches which extended about 1 m into the hangingwall. In some instances the Green Bar shale was exposed. The gully sidewalls were severely fractured in places. The resultant spalling had undermined the foundations of several packs.

Dynamic closure of the W3 panel was estimated to have had a maximum value of 100 mm. This estimate was made by observing the distance that a prop had punched into the stope hangingwall, as well as the deformation of various mine poles. The normal stope closure that had occurred prior to the rockburst was taken into account in making the estimate.

Assessment of support performance

The support system in the W3 panel consisted of backfill placed 3 - 4 m from the face, 40 ton (1 m/s) hydraulic props without headboards and pipe sticks to support the backfill bags, and packs in the area of the dyke intersection (see Figure 8.3.11). Apparently, the installation of a second line of props had just commenced at the time of the rockburst. Part of the hangingwall had remained standing in the W3 panel and formed a brow. A single Camlock prop and two poles were installed beneath the brow during the rescue effort. The quality of placement of uncemented backfill in the area was good. It was noted that a space of 1 - 2 m had been left between the backfill in the W3 panel and the timber packs lining the strike gully for the storage of pipes and equipment. The backfill bag at the top of the W2 panel was tightly filled but did not completely fill the void as the bag was too small and the hangingwall was irregular. Freshly split timber was observed in several gully packs, indicating damage during the rockburst. The timbers of several packs, particularly on the updip side of the gully, had been ejected into the gully. The foundation of the gully packs had been severely eroded in places.

Twenty-eight 40 ton props were recovered from the W3 panel and tested in the laboratory. The props were exposed to both slow and rapid yield (1 m/s) tests. Seven props were non-functional and could not be tested. Problems included water leaking through the main seal or rockburst valve, jamming of the hydraulic ram, or dirt in the filler valve. It is not known whether these props were damaged during the rockburst. Three props exhibited setting problems and could not be tested at slow yield, and were only subjected to the rapid yield test. All twenty-one props subjected to the 1 m/s rapid yield test survived without catastrophic failure. However, there was a considerable range in the force characteristics of the props. Not a single prop met the force characteristics specified for 40 ton props, although most would still have been able to absorb a large amount of energy. Two props were re-tested about a month after the initial tests, and gave results which differed considerably from the initial test. Consequently, it is difficult to infer the performance of the props during the rockburst from the results of the

laboratory tests. Those props which were marked as having 'punched' into the hangingwall did not show any unusual force characteristics; and it was not possible to determine whether or not these props had been fully closed prior to the rockburst.

Assessment of the layout

When interpreting the results of elastic numerical modelling, it must be understood that the values obtained give *global* estimates of stress and energy redistributions which may be useful for comparing and evaluating different mining options, but are certainly not the actual values expected to exist within the envelope of *fractured* rock surrounding mining excavations. For example, *face* stresses greater than 200 MPa will probably not exist, as the rock would have already failed and transferred the stress deeper into the confined rock mass. However, highly-stressed areas indicate a potential for violent failure. A back-analysis of the longwall was carried out using the MINSIM program (Chapter 11.4.4). The face positions at the time of the $M = 2.5$ event were used in order to establish the energy release rate (ERR) associated with the working faces, as well as excess shear stress (ESS) on the plane of the dyke and the major principal stress (σ_1) acting on the dyke (Chapter 3.3.3). At the time of the rockburst, the ERR was in the range 30-40 MJ/m² along most of the working faces, and approached 50 MJ/m² in the corners of the lags. The acceptable upper limit of ERR for the CLR at the mine is considered to be 40 MJ/m². High stresses ($\sigma_1 > 500$ MPa) were found updip of the strike gullies. Most significantly, very large ESS lobes ($\tau_1 > 10$ MPa over 12000 m²) were present on the plane of the dyke.

A back-analysis was also conducted of a rockburst which took place along the same dyke two years earlier, resulting in six fatalities. In this instance, part of the shaft pillar was being mined. At the time of the rockburst the ERR was generally less than 30 MJ/m², with a few localized areas in the range 40-50 MJ/m². Extremely high stresses ($\sigma_1 > 500$ MPa) occurred along the faces. ESS lobes were large ($\tau_1 > 10$ MPa over 5000 m²).

Strategies for mining through a second dyke 100 m ahead of the face were evaluated using similar numerical modelling. The ERR, ESS and σ_1 values for each of the basic layouts were compared with those operating at the time of the rockburst. The second dyke strikes N-S and dips to the east at 70°. The reef is downthrown to the west of the dyke by about 3 m. An underhand configuration (longwall orientation N30°W) was found to be most favourable, as ERR was generally < 20 MJ/m², the stress was more evenly distributed along the face and the occurrence of localized high stress zones was minimized. ESS lobes ($\tau_1 > 10$ MPa over 850 m²) were also far smaller than for other alternatives considered. This is an illustration of the principle of mining *obliquely* up to or through seismically-active geological features (c.f. Figures 2.4.1, 3.4.8).

Seismic history

The seismicity for the 10 month period prior to the rockburst is shown in Figure 8.3.12. Most of the activity occurred in the footwall extension of the dyke to the NW of the reef intersection. Two events with $M > 2$ occurred close to the current position of the W3 panel during this interval. An $M = 2.6$ event occurred 60 m in the hangingwall 10 months prior to the event, and an $M = 2.3$ event occurred 60 m in the footwall 6 months prior to the event. This would imply that the W3 panel was being mined close to a volume of rock where failure had previously occurred, which could account for some of the shear fractures observed in the W3 panel. An $M = 3.2$ event occurred on the second dyke.

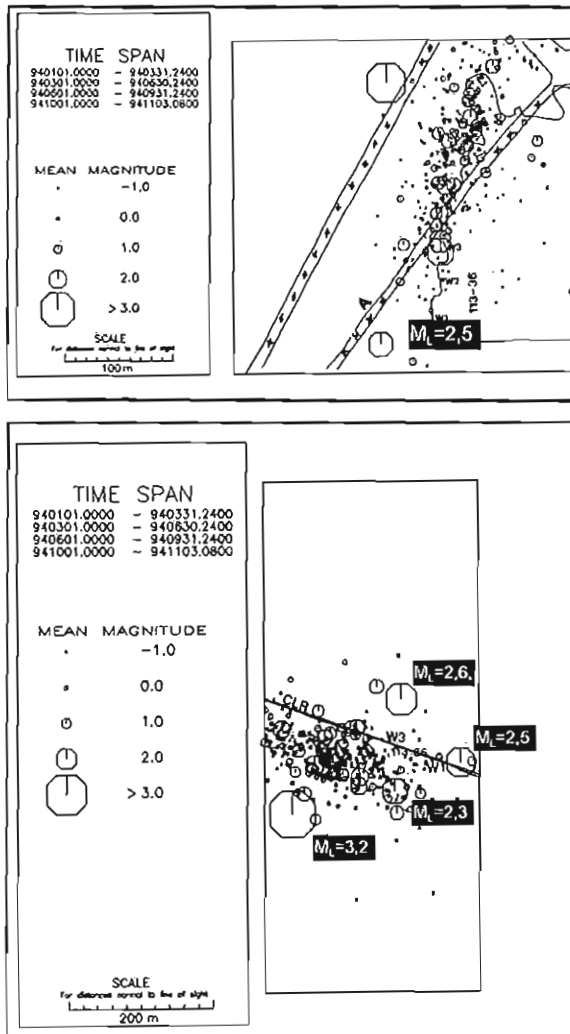


Figure 8.3.12
Plan and section showing seismicity for a 10 month period prior to the rockburst. Face positions indicated are those 5 months prior to the rockburst.

Table 8.3.2 Source parameters

EVENT SIZE			
Local magnitude:	2.5	Apparent volume	0.35 x 10 ⁶ m ³
Brune source radius	64 m	ENERGY	
MOMENT			
Seismic moment:	4.66 x 10 ¹² Nm	Radiated Energy	1040 MJ
P Moment	3.45 x 10 ¹² Nm	P Energy	12.9 MJ
S Moment	5.82 x 10 ¹² Nm	S Energy	1010 MJ
Moment Magnitude	2.4	Energy Magnitude	2.8
S Moment/ P Moment	1.7	S Energy/ P Energy	78.7
		Energy Index	0.5 (rel. to CLR events) 1.2 (rel to VCR & CLR)
STRESS		CORNER FREQUENCY	
Apparent Stress	6.7 MPa	Corner Frequency	32 Hz
Static Stress Drop	14.9 MPa	P Corner Frequency	35 Hz
Dynamic Stress Drop	14.7 MPa	S Corner Frequency	28 Hz

Rockburst mechanism

The seismic source parameters for the $M = 2.5$ event are given in Table 8.3.2: a 'slip-type' source mechanism was determined – see Chapter 9 for definition of some of the terms involved.

Conclusions

1. Part of the quartzite wedge between the A dyke and the face of W3 failed, probably owing to a stress redistribution following the $M = 2.5$ slip event occurring on the dyke. The size and geometry of the wedge is thought to have been critical.
2. Several sets of joints, partings and mining-induced extension and shear fractures occur in the W3 panel, many of which contributed to the scale of the FOGs associated with the seismic event.
3. Previous seismic events along the A dyke may have caused the rock mass in the W3 panel to be exceptionally fractured prior to the rockburst.
4. The small angle of approach (about 17°) between the longwall and the dyke is thought to have been an important factor contributing to the rockburst damage in the W3 panel. Faults and dykes are often accompanied by adjacent sympathetic features which lead to friable ground conditions; while small angles of approach imply large lobes of positive ESS, increasing the possibility of large magnitude seismic events taking place on a given geological weakness.
5. The hangingwall of the W3 panel fragmented and collapsed due to dilation of the failed quartzite wedge and seismic shaking.
6. The single row of props and poles situated against the backfill did not give adequate support to the 1 m thick hangingwall slab between the backfill and the stope face. The presence of a row of rockburst-resistant hydraulic props with load spreading headboards between the face and backfill would probably have limited the damage.
7. The uncemented backfill was generally well placed. However, the gap between the backfill and gully packs contributed to the fall out between the packs.

Recommendations

1. The mine should develop and adhere to layout strategies for mining in the proximity of geological discontinuities such as dykes and faults. Back analyses of past rockbursts should assist in the setting of empirical design criteria. For example, the use of bracket or buttress pillars to clamp the structures should be considered – Chapter 3.3.3.
2. The angle between the longwall and the dyke should also be carefully considered. Experience shows that an angle greater than 35° along the entire longwall is desirable – Chapter 3.5.2.
3. Preconditioning of seismically-active dykes is a possible control option – section 8.4.
4. Two rows of hydraulic props with load-spreading headboards should be used between the backfill and the face to give a better areal coverage in the face area. An unsupported span of more than 2.5 m between face and backfill should be avoided.
5. The backfill bags should be large enough to tightly fill the stope.
6. Backfill should be extended to the down-dip gully packs. This would increase the filling by about 5% and reduce the potential for falls of ground between the gully packs.
7. The following specific mining strategy was recommended based on the results of the back analysis and numerical modelling:

- Mineral on the W1, W2 and W3 panels of the longwall should be discontinued to create a bracket pillar around the A dyke.
- An underhand layout should be adopted to negotiate the second dyke 100 m further west. This layout gives a much improved angle between the longwall and the dyke.
- Leads and lags should not exceed 5 m to prevent the creation of highly stressed and cross-fractured areas of ground.

8.3.5 Case History 4 - Shakedown Burst Caused By Incline Shaft Pillar Failure

A rockfall in July 1995 damaged the secondary incline shaft of an East Rand mine between 32- and 33-level, about 1500 m below surface (Figure 8.3.13). Indirect evidence suggests that the rockfall was associated with a small seismic event. An event of local magnitude $M = 0.9$ occurred about 3 weeks later, causing further damage and injuries to workers rehabilitating the support in the incline shaft. No events were recorded in the 5 months prior to the first rockfall.

The upper part of the secondary incline shaft is about 10 m wide and 3 m high, and dips to the south at 45° . At the 32- to 33-level the incline shaft is situated as little as 3 m below the composite Main Reef and Main Reef Leader. Mining within 100 m of the rockfall sites last took place more than 70 years ago, but stoping had commenced a few months earlier on 38 level, about 200 m away. Overlying the damaged areas was a small 'shaft pillar' of unmined ground (blocks A, B, C, E in Figure 8.3.14).

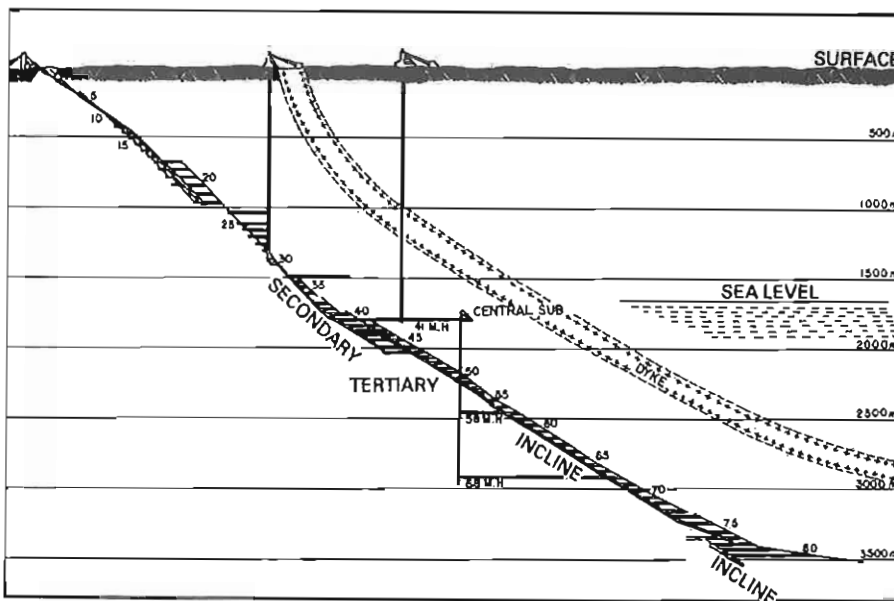


Figure 8.3.13 A seismic event occurred in the pillar adjacent to the secondary incline shaft between 32- and 33 levels, causing damage to the excavation and injuring workers carrying out rehabilitation work.

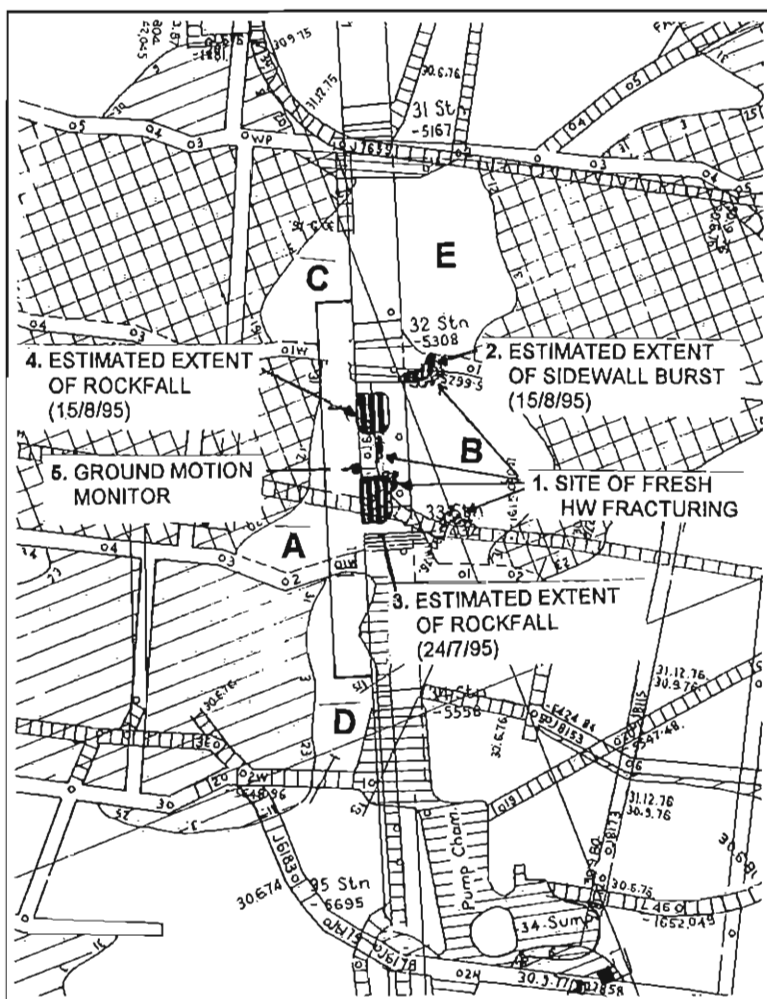


Figure 8.3.14 Plan showing the location of the rockburst damage in the secondary incline shaft between 32- and 33-level and nearby excavations.

The secondary incline shaft was being rehabilitated at the time of the first rockfall, as the rock mechanics department had previously noticed a deterioration in the condition of a section of the incline shaft. Cracks had been noted in the concrete lining on the west side, and several falls of ground had occurred. Remedial measures included barring down of loose rock, installation of new timber sets, and installation of 9 m cable anchors. Following the first rockfall, the remedial measures were upgraded, and comprised barring down of loose rock, installation of shepherds crooks, cable lacing and wire mesh, steel sets (RSJ girders at 1 m spacing, sets tied together), and 6 m cable anchors. Nevertheless, the second seismic event caused damage both to the incline shaft and the 32-level station.

Observations at the site

The first rockfall took place between the 32- and 33-level stations (Figure 8.3.14). Due to dangerous conditions in the incline shaft, it was difficult to determine the

dimensions of the fall of ground accurately. It appeared that about 0.5 m of the hangingwall had collapsed over the entire 5 m width of the conveyance section of the incline shaft. A steeply dipping discontinuity marked the up-dip limit of the fall of ground. The hangingwall was blocky and irregular, with fresh fracture surfaces present. Separation had not taken place along a bedding plane.

The support installed in the incline shaft consisted of steel and timber sets. The bulging of the western wall had caused the legs of the sets to be 'kicked out' at the time of the fall. The concrete siding on the west side was broken in several places. Apparently, fractures had developed over a period of several weeks prior to the incident, as had squeezing of the steel girders across the shaft hangingwall.

The second incident took place some 3 weeks later. The site of this fall was just below the 32-level station, with some damage also occurring in the 32-level station area. The damage appeared very similar to the first incident, in that large volumes of freshly broken rock had fallen into the incline shaft immediately below the point where the new support had been installed. The area around the 32-level reef drive (2 in Figure 8.3.14) had burst into the excavation. The hangingwall of the shaft above 32-level exhibited damage in the form of fresh cracks and small areas of freshly exposed rock. Some squeezing of the sets was also apparent. Some damage to the hangingwall of the 32-level station was noted. A cracked concrete wall in the 32-level station area provided proof that convergence had occurred.

The 32-level crosscut, which lies 15-20 m above the rockfall site, was inspected following the first rockfall. Several freshly exposed fracture surfaces were noted on the upper sidewall on the eastern side of the crosscut. The 32-level hangingwall drive to the east of the crosscut was inspected. Freshly exposed fracture surfaces were observed on the upper sidewall on the northern side of the drive within 10 m of the crosscut/drive junction. As these tunnels were not in regular use, it is not known whether these fractures formed prior to the rockfall or at the time of the fall. The dimensions of the freshly exposed surfaces ranged from 10 cm x 10 cm to 1 m x 1 m. Slabs 1-2 cm thick had spalled from the sidewall. The surfaces were glassy, with sharp angular grains still adhering to the surface. These surfaces bore no trace of rusty staining, which was otherwise ubiquitous. The 32-level hangingwall drive was inspected to the west of the crosscut and no freshly exposed surfaces were observed over a full 80 m section. Freshly exposed fracture surfaces were observed in a reef drive to the east of the 32-level station. Following the second rockburst, the 32-level hangingwall drive to the east of the crosscut was inspected again (see 1 in Figure 8.3.14). Further freshly exposed stress fracture surfaces were observed on the upper sidewall on the northern side of the drive within 10 m of the crosscut/drive junction, indicating a further increase in stress on Pillar B.

Rockburst mechanism

A seismic monitoring system was operational on the mine; however, no seismic events were recorded in the vicinity of the shaft at the time of the rockfall. The nearest geophone station is about 1 km from the rockfall site. An inspection of the seismic history indicated that only $M > 0.5$ events occurring in the vicinity of the shaft would be recorded. It was reported that audible microseismic activity was noted both prior to the rockfall and on later visits to the site, and that the banksmen had heard a seismic event at the time of the rockfall.

The following rockburst mechanism is proposed (see Figure 8.3.15).

1. The pillar (marked A in Figure 8.3.14) to the west of the incline shaft was 8-10 m wide between 32- and 33-level. This pillar had gradually lost its capacity to bear load in the months preceding the major rockfall, probably due to the evolution of stress fractures around the incline shaft and stope, exacerbated by any barring away of the hangingwall in the incline shaft adjacent to the pillar. This would tend to weaken the pillar by decreasing the width to height ratio.
2. The fracturing within the pillar caused dilation, which buckled the strata over the incline shaft downwards. This buckling caused the hangingwall to become unstable, and minor rockfalls to occur.
3. The position of fractures observed in the 32-level crosscut and drive are consistent with increased load being placed on the 22-30 m wide pillar to the east of the incline shaft (B in Figure 8.3.14) as pillar A failed or partially failed, and stress was transferred across the incline.
4. The rockfall was triggered by the failure or partial failure of pillar A. This probably gave rise to a small seismic event (with $M < 0.5$) which produced some shaking. Dilation of the pillar caused damage to the concrete sidewall of the incline shaft, caused the sets to collapse by pushing out their legs, and initiated new fractures in the hangingwall. The fall of ground followed.
5. A ground motion monitor was installed in the 32-level crosscut following the first rockfall (see 5 in Figure 8.3.14). A total of 19 seismic events were recorded in a three week period, including a large event which coincided with the $M = 0.9$ seismic event recorded on the mine seismic network at the time of the second damaging event. The accelerograms clearly showed that the seismic sources were located within a distance of 15 m to 50 m from the monitor, strongly suggesting that the pillars were the source of the seismicity (most probably pillar A in Figure 8.3.14).
6. The disposition of the rockfall and the seismicity recorded by the ground motion monitor indicates continuation of the failure of the narrow pillars to the west and east of the incline shaft.

Conclusions

1. The first rockfall was triggered by the failure of the narrow pillar to the west of the shaft (pillar A in Figure 8.3.14).
2. The barring of unstable rock from the hangingwall of the incline shaft adjacent to the pillar during resupporting had the effect of decreasing the width to height ratio of the pillar. This would have had the result of weakening the previously stable pillar. As pillar A progressively failed, the load was transferred to adjacent pillars (such as pillar B in Figure 8.3.14). Evidence for the increase in stress is provided by fresh stress-induced fractures observed in the hangingwall drive (1 in Figure 8.3.14).
3. The progressive failure of pillar A resulted in the horizontal dilation of both the hangingwall and footwall of adjacent strata, causing the hangingwall beam of the incline shaft to buckle and ultimately to collapse. It is possible that the collapse was triggered by a small seismic event with magnitude $M < 0.5$. [A similar phenomenon has been noted in bord and pillar workings, where underdesigned pillars progressively fail and dilate, causing the hangingwall to deflect and collapse into the bords].
4. The second rockfall and the seismicity recorded by the ground motion monitor indicates continuation of the failure of the narrow pillar to the west of the incline

shaft (pillar A in Figure 8.3.14). The sidewall burst of pillar B east of the incline (at 2 in Figure 8.3.14) is further evidence that stress was being transferred to the pillars to the east of the incline shaft.

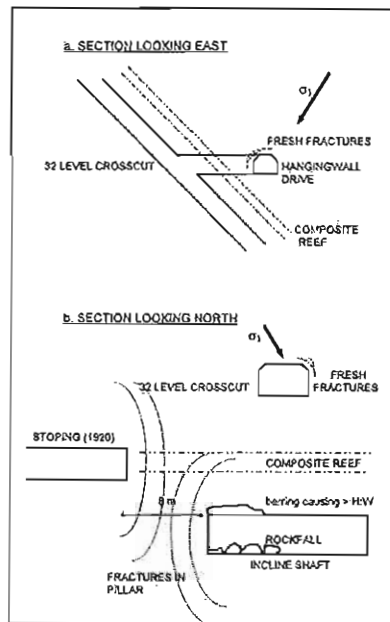


Figure 8.3.15 Diagram showing the interpreted mechanism of the rockbursts/rockfalls damaging the secondary incline shaft between 32- and 33-level.

Recommendations

Similar narrow pillars were identified elsewhere in the incline shaft. As the transfer of stress from one pillar to another is a time-dependent process, seismicity could occur in the future, particularly if further rehabilitation work is carried out. The following precautions were recommended:

1. Identify situations where the pillar abutting the incline shaft is narrow (e.g. between 33- and 34-level of the secondary incline shaft, marked D in Figure 8.3.14). Extensometers may be used to establish whether dilation of the pillar is, in fact, taking place.
2. Microseismicity in the vicinity of the narrow pillars should be monitored.
3. Inspect excavations close to the incline shaft for any fresh fractures which may indicate a change in the state of stress.
4. Inspect the shaft hangingwall for instability. Ground penetrating radar may prove useful in identifying parting planes.
5. Avoid increasing the unconfined height of the shaft pillar through barring down the hangingwall.
6. Investigate using grouts and resins to consolidate the fractured hangingwall.
7. Ensure adequate temporary support is in place when cable anchors are installed, e.g. mine poles.
8. The stability of sets in a steeply dipping excavation can be improved through the use of ties.

8.3.6 Synthesis Of Rockburst Investigations

Thirty-one rockburst investigations were conducted by the CSIR:Miningtek team during the period 1994 to mid-1998. The majority of investigations took place in the Far West Rand gold field, with several investigations in the Klerksdorp, West Rand and East Rand gold fields. In this section the findings of these investigations are synthesized in order to highlight common themes and key issues.

a) Layout and mining sequence

Guidelines and empirical design criteria, such as those to be found in this volume and its predecessors, are generally used for the design of excavations. During the course of the rockburst investigations, it was found that some of these guidelines and criteria have limitations which are not always appreciated by the rock engineering practitioners involved; nor, in some instances, are the limitations clearly expressed in the publications. The notes that follow for the most part endorse existing guidelines, but in some instances reflect a slowly evolving change in knowledge and approach.

Remnants

The mining of remnants poses particular challenges (Chapters 3.5.1, 4.6.1), as these parts of the orebody have often been left because of geological complications such as faulting, or because of damage caused by previous rockbursts. The formation of remnants should be avoided wherever possible, and underhand mining from the original raise is often the best layout in deep situations. The mining of a remnant between approaching longwall faces is inherently hazardous and must be carefully managed. The formation of rectangular or L-shaped remnants should be avoided, as the whole structure may fail in a single event. Rather, a triangular remnant should be formed and mined in a direction such that the part to be mined last is closest to the nearest large solid area, allowing the apex of the triangle to crush continuously while being subjected to a relatively stiff loading environment.

Pillar width:height ratios

If trenching is carried out owing to a sudden change in reef elevation (e.g. faulting or 'rolling'), the effective unconfined height of the remnant or pillar is increased (Figure 8.3.2). Similar remarks apply to gully pillars when sidings are not used. Calculations of the pillar dimensions using guidelines based solely on width:height ratio should be treated with caution. The influence of fracturing needs to be considered.

Face orientation

The angle between a longwall and geological features such as dykes, faults, or dominant joint sets must be carefully considered. Experience shows that an angle of at least 35° between the feature and the orientation of the longwall (as well as individual panels) is desirable - Chapters 2.4, 3.4, 3.5.2.

In situ stress

The in situ stress is an important factor in designing underground excavations, and cannot simply be assumed to be the overburden load together with a k-ratio of 0.5 - Chapters 1.3.1, 3.5.3. Observations or measurements need to be carried out to determine whether any anomalous stress states exist. In some areas, significant anomalies have been detected, with k-ratios as high as 2.0 making shallow dipping structures more susceptible to shear.

Shape of slope face

Panel length and lead/lags require trade-offs between practical production constraints and the theoretical ideal - Chapters 2.4, 3.4. A long straight face should be avoided. The use of shorter panels is recommended to limit the extent of ruptures and consequent

damage along face-parallel shears, and to facilitate escape from rockburst damaged panels. Lead lengths should generally be restricted to about 5 m. Gully headings in advance of long straight faces are particularly prone to damage. Panels lagging by large amounts are subjected to high ERRs and unfavourable fracturing, and should be supported particularly well, with a strictly enforced 'no blast if support not up to standard' regulation. The leading panels should be stopped or slowed down to remedy the situation.

Service excavations and facilities

Facilities such as the stope entrance infrastructure (timber and material bay), refuge bay and waiting place should be located away from seismically hazardous areas such as faults or dykes.

Faults/dykes

Mining in the vicinity of faults/dykes often results in increased instability, and the use of bracket/buttruss protection pillars should be considered – Chapter 3.3.3.

Slope access

The number of accessways to the face should be adequate for the rescue and rehabilitation work that may be required following a rockburst, and need to be kept clean and in good condition.

Angular unconformities

If strata are unconformable and therefore not parallel, the distortion of the stress field due to mining can increase the shear stress on bedding planes with low cohesion, contributing to the chances of a seismic event occurring.

Stabilizing pillars and abutments

Highly-stressed stabilizing pillars and abutments can experience foundation failure. The situation may be alleviated by the use of higher volumes of backfill and improved placement techniques, thereby reducing the likelihood or effect of further foundation failures. For mines without backfill systems, consideration may need to be given to changes in layout, including increasing pillar size and reducing the spacing between pillars to ensure that APS levels are kept below locally-established critical levels – Chapter 3.3.2.

b) Numerical modelling

When mining layouts are designed, the guidelines and empirical criteria are often supplemented by the calculation of Energy Release Rate (ERR) and Excess Shear Stress (ESS) using standard numerical modelling computer programs. During the rockburst investigations it was found that the fundamental assumptions of elastic modelling techniques, and the need to apply engineering judgement in the interpretation of the results, are not always appreciated by the rock engineers on the mines. Mines need to develop improved strategies for mining in the proximity of geological discontinuities such as dykes and faults using back-analyses of past rockbursts. ESS and ERR should provide empirical design criteria for specific geotechnical areas and reefs.

Extreme care must be taken in the interpretation of calculated stresses and ERR or ESS values. For example, the ratio of the average pillar stress (APS) to the uniaxial compressive strength (UCS) of the rock comprising a pillar system is commonly used as an empirical design criterion. Elastic modelling programs do not take the fracturing of the face into account, and produce unrealistically high values of stress at the edges of pillars and abutments. In reality these areas fracture and crush, shifting the load away from the face. The core of a small pillar may be subjected to greater stress than is indicated by the numerical model, and the indicated APS may not be appropriate for calculating the APS/UCS ratio. This can become critical when the pillar dimensions are small.

The mining and seismic history should be considered when assigning strength to blocks of ground. Narrow pillars, small remnants and areas that have hosted large seismic events should not be modelled as solid, but rather as failed areas incapable of bearing significant load. Moreover, footwall punching and complete stope closure may have partially relieved stresses on pillars and remnants.

Other important factors are the sizes of mesh and the window used for numerical modelling. The mesh size should be small enough to represent the local mining geometry in adequate detail, while the window size should be large enough to take into account all significant contributions to the stress in the area of interest - Chapter 11.4.

c) Tunnels and service excavations

The quality of the support system is of key importance - Chapters 6 and 7. Long-term excavations that are likely to be subjected to seismicity during their lifetimes should be supported with yielding units that can accommodate shear deformations (e.g. grouted rope anchors or cone bolts), integrated with mesh and lacing. In areas where severe shaking is expected, these supports should be supplemented with shotcrete. The collapse of large sections of tunnel may be precipitated by the failure of a single weak link. Consequently it is important that the lacing be properly clamped so that the failure of a single cable does not cause the whole system to unravel. Rehabilitation of tunnels and shafts by 'bleeding-off' the fractured rock may have unforeseen results. For example, the barring of unstable rock from the hangingwall of an incline shaft had the effect of decreasing the width to height ratio of the pillar separating the shaft from an old stope, causing the previously stable pillar to fail (see Case History 4). Furthermore, it is crucial that adequate temporary support is in place while rehabilitation is being performed. Other vulnerable situations arise when tunnels traverse faults, approach the reef intersection, or the stress regime changes owing to over- or under-stoping.

d) Gullies

Gully support

Rockburst-resistant support must be installed in gullies, especially when traversing faults and dykes. The use of low yield force support on gully edges is favoured. This can be achieved by installing packs which are initially stiff but which yield at about 1000 kN, or by bringing backfill down to the gully edge, with gaps left for storage. The integration of elongates with packs on gullies appears to show improved performance when compared to current standards. The idea of using elongates with special headboards to allow lagging across gullies also looks promising. The gully heading should be supported with rockburst-resistant support (such as rapid yielding hydraulic props with headboards) installed in the face area. See also Chapter 4.4.9.

Gullies adjacent to pillars and abutments

Gullies along pillars and abutments are particularly prone to damage, as these areas can host large seismic events and the gullies are exposed to high stresses over long distances. The support systems in these gullies needs to be especially robust, and innovative thinking is necessary. Some methods of reducing the rockburst hazard are suggested below.

- Use foam cement in the down-dip siding alongside and behind the packs to absorb the impact of the dilating rock and to maintain the integrity of the hangingwall strata.

- Use yielding tendons together with some form of areal support to pin the gully hangingwall. This type of support is more capable of accommodating shear along weak planes parallel to the hangingwall. Orient this support at right angles to the dominant fracturing.
 - Place backfill closer to the gully edge. Prevent backfill from dilating into the gully by using mesh between packs, or by using 'Filpacks'.
 - Precondition the pillar edges by drilling and blasting from the heading. This will create a fractured 'buffer', and ensure that the shear zone resulting from any foundation failure is more distant from the pillar edge.
 - The gully siding should be deep enough so that the pillar edge and the packs on the down-dip side are separated by at least a metre. This will reduce the likelihood of packs buckling due to violent dilation of rock from the pillar edge.
- Use foam cement or backfill to maintain the integrity of the hangingwall in this area.

Gully sidewalls

Gully packs sometimes collapse or are ejected during rockbursts, due to poor foundations. The sidewall may be damaged by scraping, poor blasting practice, or may have failed due to the gully packs bearing excessively high loads. Use of low yield force packs, or of gully sidewall tendon reinforcement, should be considered. See also Chapter 4.4.9.

Gullies in Carbon Leader Reef stopes

Carbon Leader Reef gullies appear to be prone to damage due to the geotechnical properties of the hangingwall strata. The Carbon Leader Reef is immediately overlain by a competent siliceous quartzite, 1,4 m to 4 m in thickness in the Carletonville area; which is in turn overlain by the Green Bar, a 1 m to 2,5 m thick argillaceous unit. Owing to the poor cohesion between the hangingwall quartzite and the Green Bar, the quartzite beam is susceptible to fracture and collapse. In some instances there has apparently been lateral motion along the Green Bar. In one case the gully had been excavated along the lower edge of the stabilizing pillar where a prominent set of mining-induced fractures orientated parallel to the edge of the pillar was present, giving rise to poor hangingwall conditions. Strike gully sidings must be mined strictly on dip so that the Green Bar contact is kept a maximum distance above the stope. The final cleaning of the siding can take place from the following down-dip panel where applicable.

e) Stopes

Support

Rockburst-resistant support such as rapid yielding hydraulic props or pre-stressed yielding elongates with loadspreaders must be installed in the face area. This is especially critical at the top and bottom of the panels where cross-fracturing exists due to the adjacent leading or lagging panel. Non-yielding support elements such as mechanical props and mine poles have very low energy absorption capabilities. Headboards should be fitted to props and elongates to limit falls of ground, especially in areas where the hangingwall is friable and prone to fragmentation. Adequate face area support must be in place during all working shifts. In high-risk special areas, new rows of support should be installed, and hydraulic props moved, after every blast. Back areas should be barricaded to prevent casual access, as these areas are prone to shake out.

Brows

It is important that horizontal confinement be applied to brows formed by falls of ground, and during the negotiation of small faults and 'rolls'.

'Rolls' in the Ventersdorp Contact Reef

Special care should be taken to support the hangingwall when mining in the vicinity of

rolls, especially when associated with bedding-parallel faulting, as the frequency of weak calcite-coated joints appears to increase in these areas and the hangingwall has a greater propensity to disintegrate when subjected to seismically-induced shaking. An additional hazard is posed by the exposure of lava in the face. The lava has a higher uniaxial compressive strength (UCS) and Young's modulus than the VCR, and can therefore store more strain energy with greater proneness to face bursting (see Figure 8.3.2).

Backfill

It must be ensured that the backfill bags are large enough to tightly fill the stope. In areas where fall out of the hangingwall has occurred, larger backfill bags or multiple bags should be used. Backfill should be extended to the gully packs. This would increase the filling by about 5% and reduce the potential for falls of ground between the gully packs.

Stoping width

Careful blasting should be practised and a conservative blast design implemented, as a reduction in stoping width will improve the effectiveness of both the face area support and the backfill (apart from giving important economic benefits).

Rapid yielding hydraulic props (RYHPs)

RYHPs were introduced almost 30 years ago, and were received with great acclaim once initial 'teething' problems had been overcome – Chapter 4.4.10. For more than two decades their performance was deemed entirely satisfactory. In recent years, however, the mining industry has become reluctant to continue using RYHPs owing to operational difficulties. The real cause of the poor performance of RYHP systems should be determined. Factors which could contribute to the high fall-out rate are:

- a) Failure to use loadspreaders. In highly fractured ground, the relatively small diameter of the end of the prop or extensions can punch a few millimetres and thus drop load. A similar effect is obtained from setting on a poorly cleaned footwall.
- b) Pump pressures incorrect. This can be caused by low air or water supply to the pump, or dirty filters.
- c) Not allowing the pump to stall properly when setting a prop.
- d) Extensions not seated properly.
- e) Valves and seals faulty, and other symptoms of inadequate maintenance.
- f) Inadequacies in staff training, supervision and motivation.

f) Seismicity

A mine-wide seismic network (yielding locations with an accuracy better than 50 m) should be installed on all mines which experience rockbursts to facilitate the identification of hazardous areas, and to aid in the back-analysis of rockbursts – Chapter 9. The seismicity data should be carefully analysed to identify which parameters are most useful as indicators of increased *rockburst hazard*. [However, the reliable and timeous prediction of rockbursts remains a remote possibility at this stage].

g) Strong ground motion

Observations of co-seismic closure and ejection velocities provide useful parameters for the design of support. In one instance (see Case History 2) the mass of an ejected block and the evidence of a failed rebar enabled a minimum ejection velocity of 1.9 m/s to be estimated. In most of the investigated rockbursts, however, the dynamic closure appeared to be considerably less than the capacity of the support systems, and the bulk of the damage was due to disintegration of the rockwalls between support elements, rather than failure of the elements themselves. This illustrates the importance of determining the stable 'dynamic span' for the support system and geotechnical area.

It may be true, however, that if the containment support such as mesh and lacing is improved, a much greater dynamic load will be imposed on the tendons or other support units and their inadequacies could then become more evident.

h) Preconditioning

Several of the rockburst investigations were conducted at sites where preconditioning was being implemented. These investigations supported the view that preconditioning reduces the hazard of *face bursts*. It is important, however, that production personnel adhere to the preconditioning guidelines. As the effectiveness of preconditioning is believed to diminish with time, intervals between face-parallel preconditioning blasts should be based on the elapsed time, not merely on the face advance.

8.3.7 Concluding Comments

Why does the severity of rockburst damage vary so much? No single, simple answer was found to the question. Probably the most important factors are *variations* in the condition and stressing of the rock mass, and the failure of inadequate support systems.

Nor is there an easy, instant solution to the rockburst hazard. Given the current methods of mining, the most important steps to be taken to reduce the hazard would seem to involve frequent inspections of working places by personnel able to identify changes in the rock mass condition and to recommend and implement appropriate changes to layout and support systems; thorough analysis of seismic data; discipline in ensuring that support is always up to standard and that the stope support system is as close to the face as possible; and adherence to sound layouts regardless of the demands of production.

8.4 ROCKBURST CONTROL: PRECONDITIONING

Preconditioning techniques offer fairly clear-cut means to reduce the hazard of face strain bursts, notably in the extraction of specially hazardous areas such as remnants or pillars. The rock mass ahead of a stope face is subjected to extremely high abutment stresses, which result in the complex network of fractures observed underground (Figure 8.4.1). Movement along the fractures and parting planes leads to partial stress relief as well as to enhanced closure of the hangingwall and footwall in the stope. However, stable sliding of blocks against one another may be inhibited by the presence of asperities, resulting in the accumulation of strain energy ahead of the stope face. As the face approaches such a 'lock-up', further movement may be triggered. If sufficient energy is stored in the rock mass, it may be suddenly released in the form of a strain burst, violently ejecting a metre or more of face rock into the working area.

8.4.1 Mechanism of Preconditioning

Preconditioning is intended to prevent the accumulation of strain energy ahead of the working face or, at least, control its release. The gas and shock generated by a blast within the fracture zone can remobilize the blocks by shearing through asperities that were causing any lock-ups. Strain energy release is facilitated by the stable sliding

of blocks past one another, thus reducing the risk of face bursting during the production shift. The stress redistribution away from the working face, due to a well-executed preconditioning blast, provides a low-stress 'cushion' ahead of the stope face which is able to absorb energy from more distant events. However, stress redistributions resulting from seismicity and time-dependent deformation of the rock mass can result in the reloading of the face area, if too much time is allowed to pass before mining the preconditioned zone. There are two primary configurations for carrying out preconditioning in tabular stopes, termed face-parallel and face-perpendicular. Results from field experiments are described below.

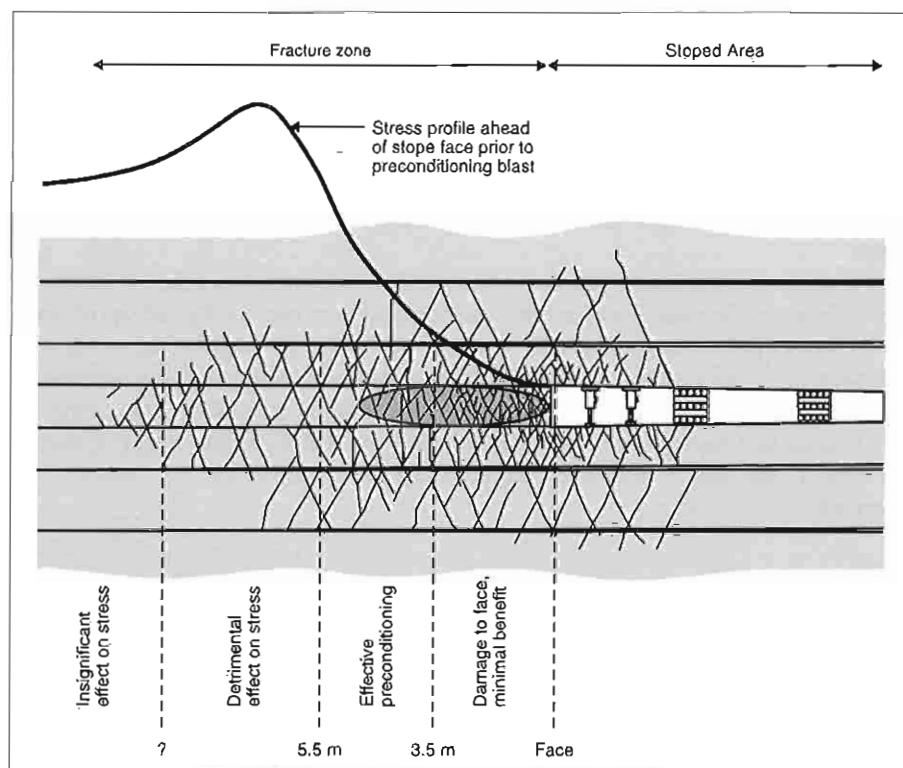


Figure 8.4.1 Conceptual diagram showing the effects of positioning an 89 mm diameter preconditioning hole within the fractured rock mass at various distances ahead of the stope face.

8.4.2 Face-Parallel Preconditioning

Face-parallel preconditioning, which appears to be the most effective of the available methods, does impose certain constraints on the stope layout to accommodate the drilling of the necessary preconditioning holes. As these need to be drilled parallel to the face (and angled slightly up-dip to facilitate drainage), an overhand mining configuration was adopted in the field experiment (see Figure 8.4.2). Preconditioning holes were drilled slightly below the reef plane, and were collared in cubbies cut from the strike gully. It was found that individual panels should not exceed the length that can

be drilled in one shift, and in this case the maximum face length was 20 m. The best position of a preconditioning hole was established to be at the limit of the 'preconditioned zone', and no fresh mining should take place beyond that point. Once a panel had mined to this limit, another preconditioning blast was taken and the cycle continued. A great deal of experimentation was required to define the area ahead of the stope face in which the preconditioning blast would provide adequate results. For an 89 mm diameter hole, the best results (for the conditions encountered in the project stope) were obtained when the hole was positioned between 3.5 m and 5.5 m ahead of the face (Figure 8.4.2). This was still within the fracture zone, but closer to the fracture front than to the face.

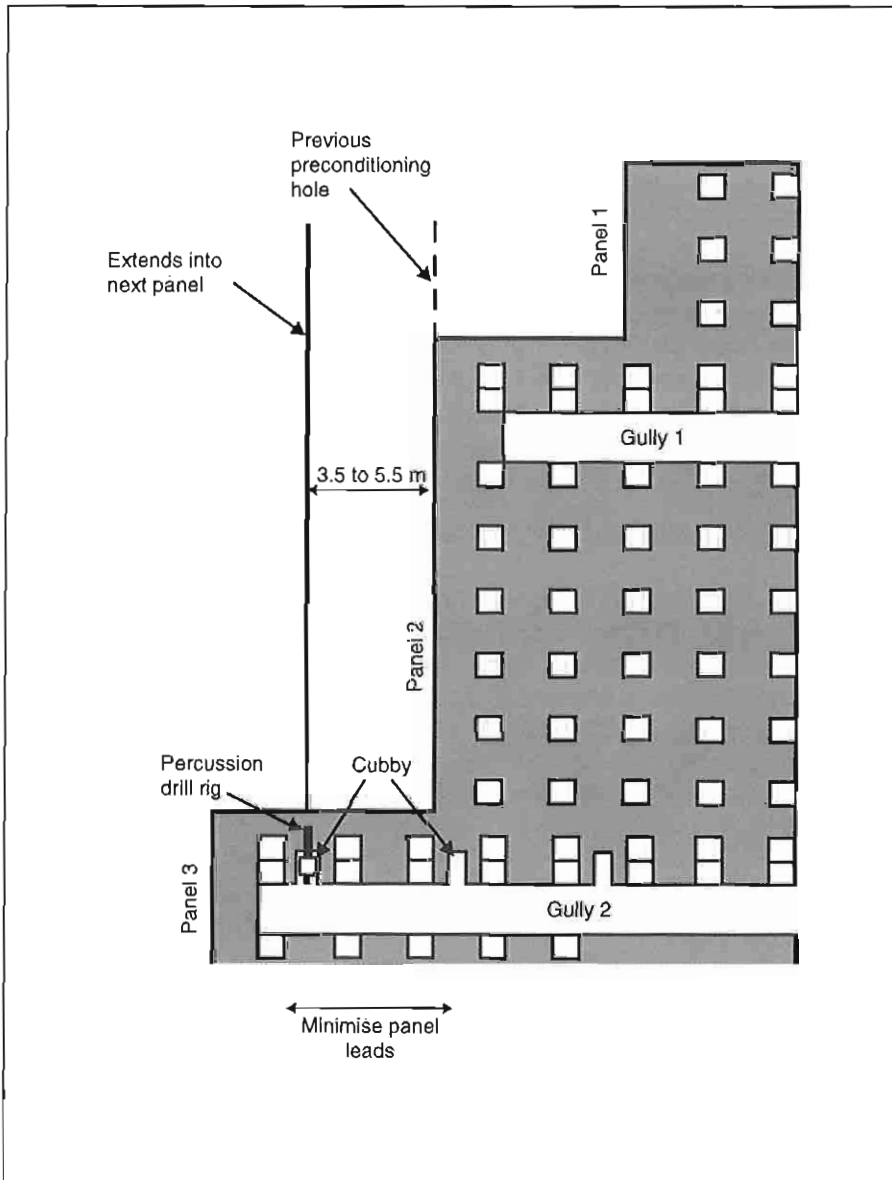


Figure 8.4.2 Face-parallel preconditioning

The seismic expression of preconditioning includes the blast event itself, an increase in the micro-seismicity rate induced by the blast, and the frequent occurrence of larger seismic events triggered by the blast. The local magnitude of the recorded 'blast event' was typically in the order of $M = 1.0$ for 100 kg of an emulsion explosive. On occasions, significantly larger events occurred simultaneously with the blast, and are considered to indicate the release of additional stored strain energy. Smaller 'blast events' were recorded from inadequate preconditioning blasts. One important observation relating to the effect of preconditioning on the rock mass ahead of the stope face concerns the damage patterns associated with larger seismic events at the site. While a number of large ($M > 2$) seismic events occurred within a few tens of metres of the faces, very little damage was observed in the face area, although substantial damage was at times evident in the access ways. These repeated observations indicate that the preconditioning was effectively creating a 'buffer' zone of destressed material ahead of the faces, which was capable of absorbing substantial seismic energy emanating from large seismic events and thus preventing damage to the working areas in the immediate vicinity of the faces.

Effective preconditioning blasts were found to induce stress transfer away from the preconditioned area (as shown by the spatial migration of subsequent seismicity towards unpreconditioned ground), and to facilitate the release of stored strain energy from the rock mass ahead of the preconditioned face by relatively larger events occurring simultaneously with the blast, or within several hours of the blast but while the working area was evacuated.

8.4.3 Face-Perpendicular Preconditioning

Although face-parallel preconditioning is well suited to the mining of long and narrow strike pillars and remnants, it is difficult to implement in a normal deep-level longwall production environment without imposing delays on the mining cycle. A face-perpendicular preconditioning technique has been developed which partially overcomes these problems. Preconditioning holes (typically 3 m in length) are drilled in the centre of the reef at a spacing of 3 m or less in addition to the normal production holes (Figure 8.4.3). They are detonated slightly earlier than the production round.

In the year following the introduction of this preconditioning technique, no face that was being currently preconditioned had been subject to strain bursting or suffered damage as a result of neighbouring seismic activity. Preconditioned stopes also exhibited improved hangingwall conditions when compared to adjacent normal stopes, and the rate of face advance improved by approximately 25 %. Fracture mapping was carried out prior to the commencement of preconditioning, and continued during preconditioning. The mapping demonstrated that no new fractures were introduced, confirming the theory that the preconditioning mechanism involves slip on existing fractures rather than the development of new fractures. However, the incidence of shallow-dipping fractures was reduced and a more regular hangingwall was produced.

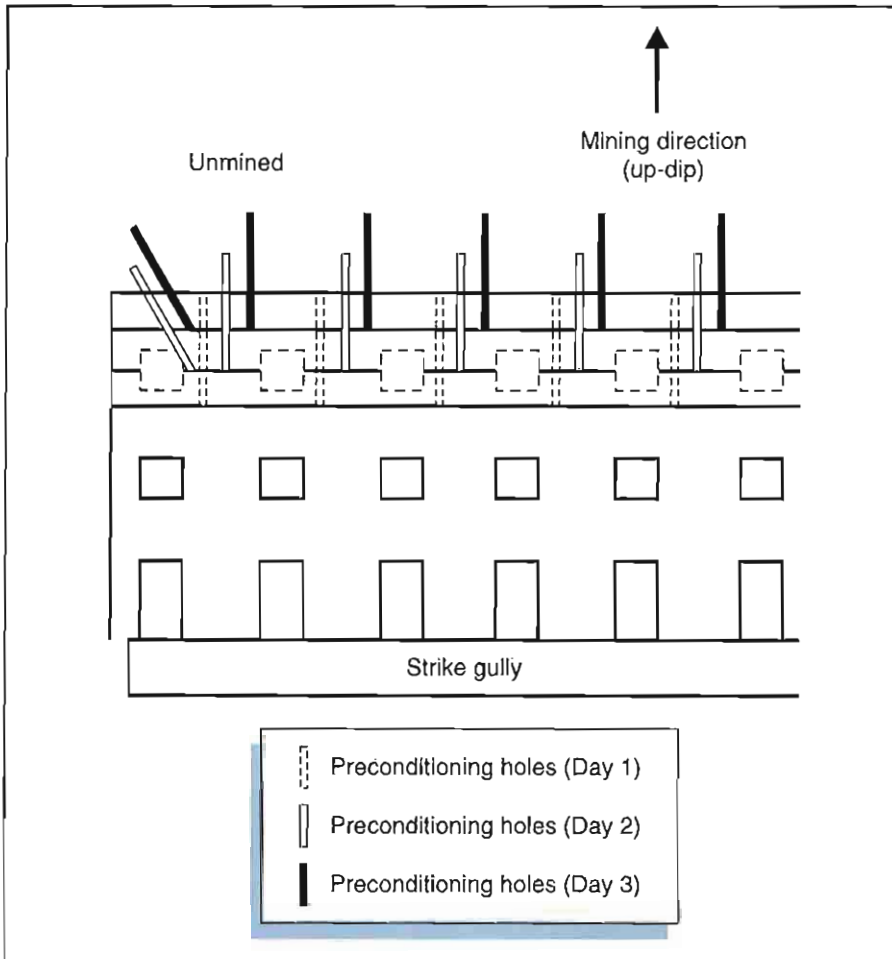


Figure 8.4.3 Face-perpendicular preconditioning

8.5 ROCKBURST CONTROL: PREDICTION OF LARGE ROCK MASS INSTABILITIES

In the published literature, success rates as high as 80% have been claimed for the prediction of large potentially damaging events on South African gold mines. However, the criteria used to calculate the 'success' of a prediction need to be carefully noted, as they may be less stringent than those generally used in earthquake seismology, where the following are used to assess the success of a prediction: location to within $1/2$ rupture length, size to within $1/2$ rupture length (or M to within 0.5), time to within 20% of recurrence time, and finally, demonstrated probability (in most cases the ratio of successes to the sum of successes and false alarms). For example, in the case here an 80% success rate was claimed, neither the time nor the magnitude of the forthcoming events was predicted – the limitation was to spatial prediction only.

There is a fundamental dilemma inherent in earthquake (and rockburst) forecasting which demands that a rigorous statistical approach be adopted in assessing 'success'. Suppose that seismological measurements indicate that an event of a certain magnitude will occur during a certain period of time. Presumably the area is seismically active (or the study would not have been initiated in the first place), so a certain probability exists that an event will occur during the predicted period. Thus, the occurrence of an event cannot be taken as decisive proof that the methods used to make the prediction are correct, and that they will succeed on future occasions. Of course, if a firm prediction is made and nothing happens, that should be taken as partial proof that the method is invalid.

A seismic warning system may contribute towards safety simply by raising the level of awareness of seismic hazards, and motivating the implementation of actions such as increasing the size or number of stabilizing pillars, changes in mining sequences and directions, and a review of mining strategies in general. Nevertheless, the many failures of seismic warning systems to predict the time and location of large damaging events, even in retrospective analyses using comprehensive seismic data sets, should temper expectations of reliable warnings with current technology and current understanding of seismic source mechanisms. Research in this important field is, nevertheless, ongoing.

Moreover, in the field of quantifying relative **seismic hazard** (as opposed to direct warning of incipient damaging events), the positive role of seismic monitoring and analysis is undisputed.

9.1 SEISMIC EVENT, SEISMICITY AND STATE OF THE ROCK MASS

Mining excavations induce elastic and then inelastic deformation within the surrounding rock mass. The elastic strain energy accumulated in a portion of the rock mass may be gradually unloaded due to the passage of mining, or it may be released gradually or suddenly during the process of inelastic deformation.

A **seismic event** is a sudden inelastic deformation within a given volume of rock, i.e. a seismic source, that radiates detectable seismic waves. The amplitude and frequency of seismic waves radiated from such a source depend, in general, on the strength and state of stress of the rock, the size of the source of seismic radiation, and on the magnitude and the rate at which the rock is deformed during the fracturing process.

A seismic event is considered to be described quantitatively when, apart from its timing t and location $X = (x, y, z)$, at least two independent parameters pertaining to the seismic source namely seismic moment M_0 (which measures coseismic inelastic deformation at the source), and either radiated seismic energy E or stress drop $\Delta\sigma$, are determined reliably.

Seismic waveforms do not provide direct information about the absolute stress, but mainly about the strain and stress release at the source. However, the source of a seismic event associated with a weaker geological feature or with a softer patch in the rock mass yields more slowly under lower stress, and radiates less seismic energy per unit of inelastic coseismic deformation, than an equivalent source within strong and highly stressed rock. Therefore, by comparing radiated seismic energies, or stress drops, of seismic events with similar moments one can gain insight into the stresses acting within the part of the rock mass affected by these events - see Figure 9.1.1.

A seismic system can measure only that portion of strains, stresses or rheology of the process which is associated with recorded seismic waves. The wider the frequency and amplitude range and the higher the throughput of the system, the more reliable and more relevant the measured values of these parameters become.

Having recorded and processed a number of seismic events within a given volume of interest ΔV over time Δt , one can then quantify the changes in the strain and stress

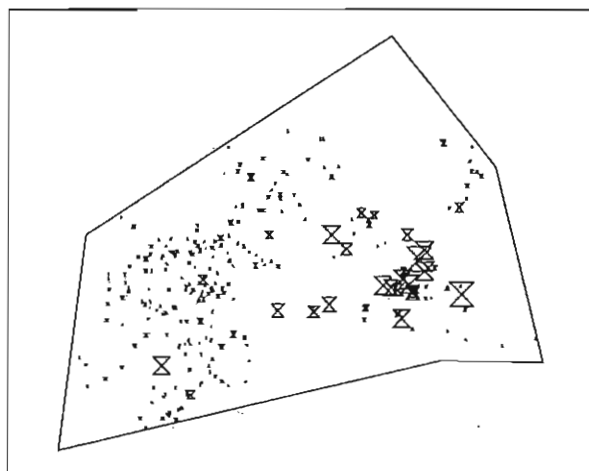


Figure 9.1.1 Apparent stresses of selected seismic events with equal local magnitudes $m = 1.0$ in a South African gold mine. The ratio of the highest to the lowest apparent stress on this figure is nearly 100. ('Apparent stress' is a measure which is approximately proportional to stress drop $\Delta\sigma$.)

regimes and in the rheological properties of the rock mass deformation associated with the seismic radiation.

This presents an opportunity to confirm the results of numerical modelling of the design process. In numerical modelling, the usual assumption of the same elastic constants within a given volume make strain ϵ and stress distribution equivalent, since $\sigma = \text{constant} \cdot \epsilon$. However, seismically-inferred stress and strain changes are independent. Seismic strain associated with seismic events in a given volume is proportional to seismic moment $\epsilon_s \propto \Sigma M_0$, and seismic stress is proportional to the ratio of seismic energy to seismic moment $\sigma_s \propto \Sigma E / \Sigma M_0$. Thus, contours of seismic strain and seismic stress may be qualitatively different, reflecting differences in stress regime and/or rock mass properties - see Figure 9.1.2 as an example. It is the difference between the modelled and seismic stress/strain distributions that needs to be explained and reconciled with the design during the mining of an area.

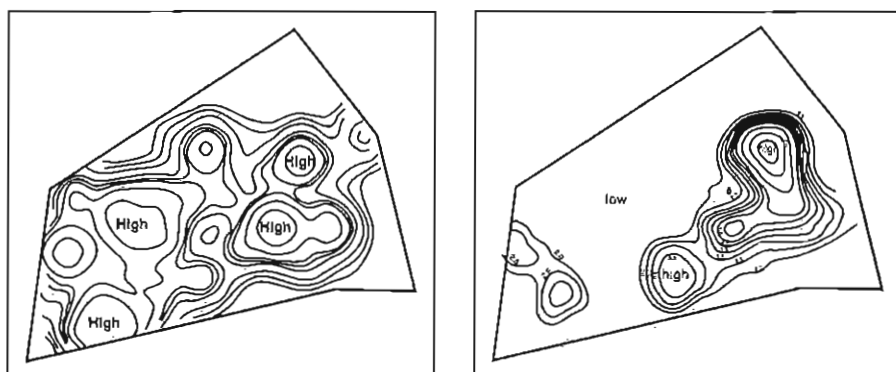


Figure 9.1.2 Contours of seismic strain (left) and seismic stress (right) for seismic events of all magnitudes in the area shown in Figure 9.1.1. Note the qualitative difference between the distributions of these seismic strains and stresses.

9.2 OBJECTIVES OF SEISMIC MONITORING IN MINES

In general, routine seismic monitoring in mines enables the *quantification* of exposure to seismicity, and provides a logistical tool to guide the effort into prevention, control and prediction or warning of potential rock mass instabilities that could result in rockbursts. One can define the following specific objectives of monitoring the seismic response of the rock mass to mining:

- * **Location of Potential Rockbursts:** To alert management by indicating the locations of potential rockbursts associated with intermediate or large seismic events and to assist in possible rescue operations - it is important then to monitor the locations of associated aftershocks.
- * **Prevention:** To confirm some of the assumptions and parameters of the design process and to enable its continuity while mining. Specifically, it is important to confirm the critical assumptions of numerical modelling which have relevance to seismic hazard. For example, small changes in the orientation of and friction on a fault may considerably affect the predicted distribution of shear stresses acting on that structure. This assists in guiding preventative measures, e.g. corrections to the designed layout, sequence of mining, given rate of mining, support strategy, etc.
- * **Control:** To detect spatio-temporal changes in seismic parameters, e.g. an increase in the number of intermediate and larger size events, a change in their time of day distribution, an increase in seismic diffusion, a degree of acceleration in seismic deformation and/or a decrease in seismically inferred stress - and relate these changes to the stability of deformation within the volume of interest. This would facilitate and guide control measures, e.g. managing workers' exposure to seismicity at different times of day, a temporary slowdown or suspension and then resumption of mining in a given area, and/or the timing and desirable location of preconditioning and triggering blasts.
- * **Warnings:** To detect unexpected strong changes in the spatial and/or temporal behaviour of seismic parameters, or certain defined characteristic patterns that could lead to dynamic instabilities affecting working places. This would facilitate warnings to manage the exposure to potential rockbursts.
- * **Back-analysis:** To improve the efficiency of both the design and the monitoring processes. Specifically important is thorough seismic and numerical modelling back-analysis of large instabilities, even if they did not result in loss of life or in considerable damage. Back-analysis of seismic rock mass behaviour associated with pillars, backfill, different mining layouts, rates and ways of excavating, etc, is an important tool in the quest for safer and more productive mining. It is desirable therefore to maintain a database of seismicity, i.e. times, locations, magnitudes, seismic moments, radiated energies, sizes and stress drops for all seismic events recorded. In addition, the availability of waveforms of the seismic events recorded a few months prior to a large event or rockburst and located within a few source diameters of that event would assist in back-analysis and research.

A quantitative description of seismic events and seismicity is considered necessary, although not sufficient, to achieving the above objectives.

9.3 LOCATION OF SEISMIC EVENTS

The location of a seismic event is assumed to be a point within the seismic source that triggered the set of monitoring sites used to locate it. The interpretation of a location, if accurate, depends on the nature of the rupture process at the source - if a slow or weak rupture starts at a certain point, the closest site(s) may record waves radiated from that very point while others may only record waves generated later in the rupture process by a higher stress drop patch of the same source. One needs to be specific in determining the arrival times if the location of rupture initiation is sought, otherwise the location will be a statistical average of different parts of the same source. A reasonably accurate location is important for the following reasons:

- * to indicate the location of potential rockbursts;
- * all subsequent seismological processing, e.g. seismic source parameter and attenuation or velocity inversion, depends on location;
- * all subsequent interpretation of individual events depends on location, e.g. events far from active mining, close to a shaft or, in general in places not predicted by numerical modelling, may raise concern;
- * all subsequent interpretation of seismicity, e.g. clustering and specifically localization around planes, migration, spatio-temporal gradients of seismic parameters and other patterns are judged by their location and timing.

Location error depends on the accuracy of the data. Table 9.3.1 lists the major aspects of the data and their minimum precision required for accurate location.

Table 9.3.1 Data precision required to achieve acceptable location accuracy

Parameters Affecting Location Accuracy (Seismic site = set of sensors with the same co-ordinates)	Recommended Minimum Precision
Common time among seismic sites in the network	500 μ s
Arrivals of P and/or S-waves at site	500 μ s or one sample
P and/or S-wave velocity model	7.5%
Site coordinates	1 m
Sensors orientation at site, used to constrain the location by direction(s) or azimuth(s) of recorded waveforms; also used in seismic moment tensor determination	5 degrees
Number of seismic sites	at least 5
The distribution of sites with respect to the position of the event to be located, e.g. as measured by the normalised orthogonality QC between straight ray paths from the hypocentre to the sites, $QC = 0.3873 \sqrt{n_s} [\det(C)]^{1/3}$, where $C = \begin{vmatrix} \sum \cos^2 \alpha_i & \sum \cos \alpha_i \cdot \cos \beta_i & \sum \cos \alpha_i \cdot \cos \gamma_i \\ \sum \cos \alpha_i \cdot \cos \beta_i & \sum \cos^2 \beta_i & \sum \cos \beta_i \cdot \cos \gamma_i \\ \sum \cos \alpha_i \cdot \cos \gamma_i & \sum \cos \beta_i \cdot \cos \gamma_i & \sum \cos^2 \gamma_i \end{vmatrix}$ and $\alpha_i, \beta_i, \gamma_i$ are directional angles between the hypocentre and the i -th site; Σ runs over the number of sites n_s	0.3

The location depends also on the numerical procedure adopted to solve the system of nonlinear site equations. The denser the network and the more accurate the data, the smaller is the influence of the numerical procedure. With high quality data from at least 5 sites of reasonable configuration, the location error may be reduced to less than 3% of the average hypocentral distance (AHD) of the sites used.

In the case of the velocity model not being known adequately, or if velocities change significantly with time, one can attempt to improve the location by the arrival time difference method, also known as 'master event' location or relative location. This procedure requires an accurately located master event (e.g. blast), in the proximity of the event to be relocated, that has reliable arrival times at sites used in the relocation procedure. It is inherently assumed here that the velocities of the seismic waves from the master event to the sites and those from the target event are the same. Since this is not always the case, it is important that the two events should be close to each other; less than 10% of average hypocentral distance would be a good rule of thumb.

Since the source of a seismic event has a finite size, the attainable location accuracy of all seismic events in a given area should be within the typical size of an event of that magnitude which defines the sensitivity of the seismic network for that area, i.e. the minimum moment-magnitude, m_{min} , above which the system records all events with sufficient signal to noise ratio (SNR). The table below gives the recommended location accuracy for different network sensitivities associated with different objectives of monitoring. Approximate source sizes are quoted for reference.

Table 9.3.2 Recommended location accuracy required to achieve specific monitoring objectives

Objective of Monitoring:	Location	Prevention		Control		Warnings	
Network sensitivity (m_{min})	1.0	0.5	0.0	-0.5	-1.0	-1.5	-2.0
Desired minimum location accuracy [m]	100	75	40	20	15	10	5
Approximate source sizes [m] at stress drops between 0.1 to 0.5 MPa	65-110	35-65	20-35	12-20	6-12	4-6	2-4

9.4 QUANTIFICATION OF SEISMIC SOURCES

Seismic events can routinely be quantified by the following parameters derived from recorded waveforms:

- * time of the event t
- * location $X = x, y, z$
- * seismic moment M_0 and its tensor, which defines the overall *direction* of principal stresses acting at the source and the nature of the coseismic *strain change* in terms of its isotropic and deviatoric components
- * radiated seismic energy E , and/or seismic stress drop $\Delta\sigma$
- * characteristic size of the event l .

The routine estimates of seismic moments and radiated seismic energies from waveforms are relatively inaccurate; with uncertainties, as measured from the scatter of

processed data around the model, from 50% for well behaved waveforms to over 100% for complex ones (Figure 9.4.1). However, the variation in radiated seismic energy (or stress drop) of seismic events with similar moments occurring in different stress and/or strain regimes at the same mine is considerably greater than the uncertainty in measurements and the error propagation in processing - see Figure 9.1.1. Thus, while these uncertainties adversely influence the resolution obtainable, they should not prevent the quantitative interpretation and comparison of seismic strain and stress changes between different time intervals and/or between different areas covered by the same seismic system.

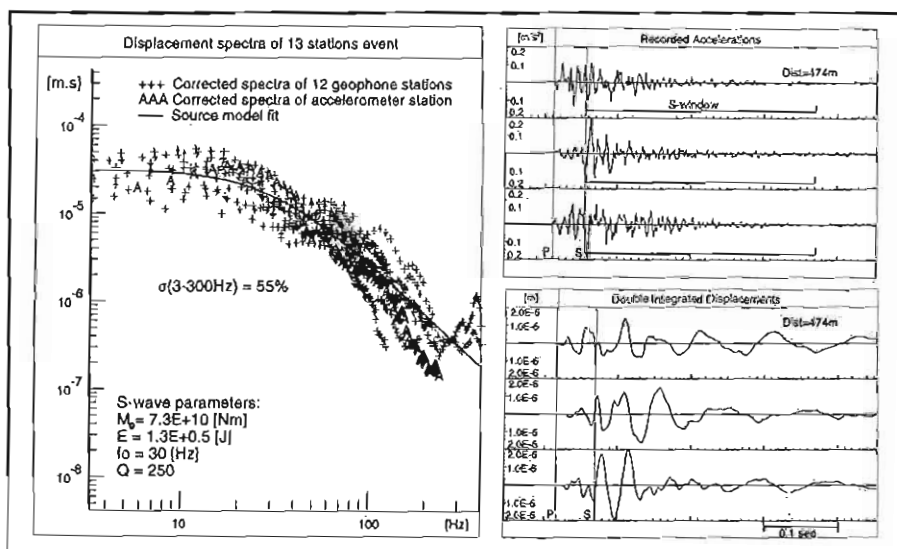


Figure 9.4.1 Left shows the stacked instrument, distance and Q-corrected S-wave spectra derived from waveforms recorded at 13 sites associated with a seismic event of $m = 1.1$. Uncertainty, as measured by standard error σ , is calculated in the frequency range 3-300Hz. Three-component waveforms of recorded accelerations with marked S-windows for spectral calculations and double integrated displacements, are shown on the right.

Seismicity is defined here as a number of seismic events, occurring within a given volume ΔV , over a certain time Δt . Seismicity can be quantified using the following four largely independent quantities:

- * average time between events $\bar{t}$
- * average distance, including source sizes, between consecutive events $\bar{X}$.
- * sum of seismic moments ΣM_0 and
- * sum of radiated energies ΣE .

From these four basic quantities one can derive a number of parameters including seismic strain ϵ_s , its rate $\dot{\epsilon}_s$, seismic stress σ_s , relative stress $\bar{\sigma}_s$, seismic stiffness K_s , seismic viscosity η_s , seismic relaxation time τ_s , seismic Deborah number De_s , seismic diffusivity D_s or d_s and seismic Schmidt number Sc_s , that describe the statistical properties of coseismic deformation and associated

changes in the strain rate, stress and rheology of the process - see Glossary of Terms for general descriptions. Table 9.4.1 lists the major aspects of data used in source parameter calculations, and the minimum precision required for reasonable results.

Table 9.4.1 Data precision required for source parameter calculations.

Parameter Affecting Source Parameter Calculation	Recommended Minimum Precision/Value
calibrated, resonance free frequency range $\pm 3\text{dB}$	$f_{\min} = 0.5f_0(m_{\max})$
$m_{\max}$ - the maximum magnitude event to be measured	$f_{\max} = 5f_0(m_{\min})$
$m_{\min}$ - the magnitude that defines sensitivity of the network	
f_0 - the corner frequency of the indicated event	
geophone natural frequency	5%
geophone damping factor	5%
geophone sensitivity	5%
accelerometer sensitivity	5%
number of sites	5 x 3 component each, or 3 x 3 comp. plus 6 single comp.
location accuracy	5% of AHD
hypocentral distance	$> \lambda = \text{wave velocity} / f_0$
$\text{SNR} = A_{\max}/\text{pretrigger noise level}$	10
P and S wave velocities	7.5%
P and S wave attenuation & scattering Q	20%
rock density at the source	10%
window length for source parameters	$4 \cdot T(A_{\max})$
$[T(A_{\max}) = \text{period associated with maximum amplitude on velocity waveforms}]$	
Uncertainties between the observed displacement spectra, corrected by the average radiation pattern, and the model.	75%

9.5 SEISMIC HAZARD AND SEISMIC EXPOSURE

In general, seismic hazard relates to the potential for strong ground motion resulting from the occurrence of seismic events. Seismic hazard is defined as the probability of occurrence of a seismic event or ground motion equal to or exceeding a specified level, within a given period of time.

The general procedure in evaluating seismic hazard includes the determination of the volumes that produce seismicity, estimating the recurrence times of seismic events of different magnitudes and, taking into account the local attenuation of ground motion, and site effects, computing the probability of exceeding a given level of ground motion for different time intervals. The results may be presented as a table of probability of each level of ground motion for a given site, or as contours of different levels of ground motion at a given level of probability. Seismic hazard estimates in mines are frequently limited to probabilities of occurrence, or recurrence times, of seismic events above certain magnitudes $\bar{I}(\geq m)$, so the relative exposure to seismicity and seismic risk can be quantified.

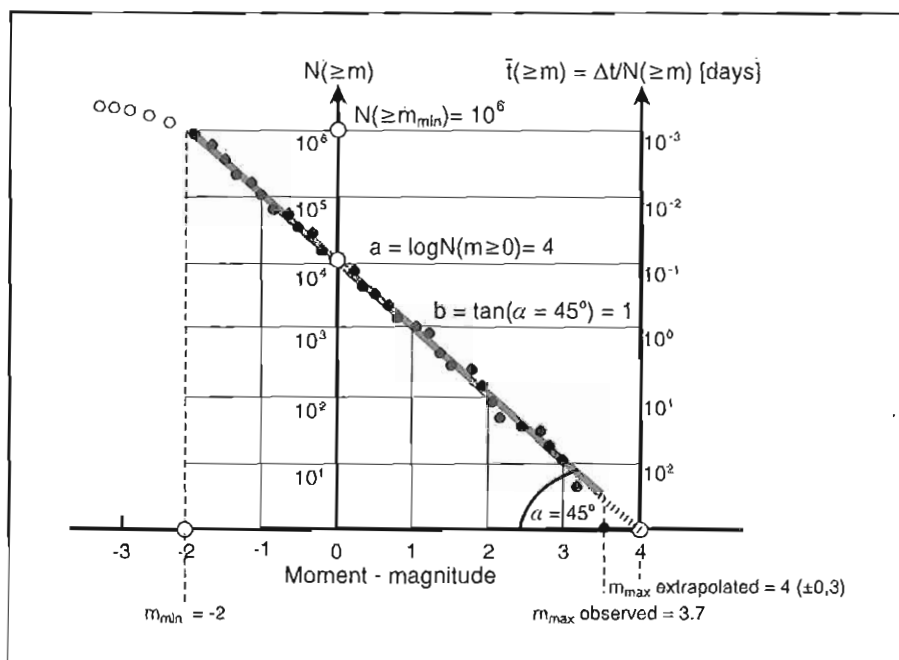


Figure 9.5.1 An illustrative plot of cumulative frequency – magnitude, with $m_{min} = -2$, $a = 4$ and $b = 1$, for the volume ΔV over the time period $\Delta t = 1000$ days. Open dots below $m_{min} = -2$ denote data points below the network's sensitivity that should not be used in parameter estimation.

The probabilistic recurrence times can be derived from different modifications of the empirical Gutenberg-Richter relation describing the frequency-magnitude distribution of small and intermediate size earthquakes,

$$\log N(\geq m) = a - bm,$$

where $N(\geq m)$ is the expected number of events $\geq$ magnitude m , and a, b are constants - see Figure 9.5.1. The Gutenberg-Richter relation implies a power-law event size distribution (further implying the absence of a characteristic event size), and as a consequence, puts no limit on the maximum earthquake magnitude. Thus if the distribution of very large earthquakes is also a power-law, then it must have an exponent (b value) that is larger than that for smaller ones: i.e. the straight-line relationship must eventually bend downwards at very large magnitudes.

The mean recurrence times can be calculated as $\bar{t}(\geq m) = \Delta t / N(\geq m)$, where Δt is the period of observation. The one largest seismic event, called here m_{max} , would have a magnitude that corresponds to $N(\geq m_{max}) = 1$, or $\log 1 = a - bm_{max} = 0$, thus

$$m_{max} = m_{min} + (1/b) \log N(\geq m_{min}), \text{ with uncertainty } \delta m_{max} = \pm 0.3 / b$$

Parameter a measures the overall occurrence rate and, for the same b , scales with the rate of rock mass seismic deformation $\dot{\epsilon}_s$ as $a = \log \dot{\epsilon}_s + \text{constant}$. The parameter b is controlled by the distribution of events between the higher- and lower-magnitude ranges and, for large and well distributed data sets, can be estimated from $b = 0.43 (\bar{m} - m_{min})^{-1}$ with uncertainty $\delta b = \pm b / \sqrt{n}$, where $\bar{m}$ is the mean of the observed magnitudes and n is the number of observations. Both parameters influence derived

recurrence times, therefore it is essential that their values and variances be determined accurately. The basic assumptions made when calculating the probabilistic recurrence times are that $N(\geq m_{\max}) = 1$ and the parameters a and b do not change significantly with time and within the selected volume ΔV .

The estimated number of seismic events within the magnitude range m_1 and m_2 , where $m_1 < m_2 \leq m_{\max}$, is $N(m_1 \leq m \leq m_2) = N(\geq m_1) - N(\geq m_2)$, and the cumulative moment release ΣM_0 by all these events can be estimated from

$$\Sigma M_0 = b \cdot 10^{a+9.1} (10^{m_1(1.5-b)} - 10^{m_2(1.5-b)}) / (1.5-b), \text{ for } b < 1.5.$$

In the long term, the sum of seismic moments is proportional to the volume mined V_m , $\Sigma M_0 \approx G V_m$. Thus, for a given V_m , the higher the value of a and b in the Gutenberg-Richter relation, the lower the $m_{\max}$ is likely to be.

The specific model of the frequency-magnitude relation and the numerical procedures used in its parameter estimation should take account of the upper limit of possible maximum magnitude in the area of interest, slow temporal changes in b , and the fact that magnitudes are not continuous but discrete grouped quantities determined, in mines, with an accuracy of 0.2 units on average.

If one assumes that the time of rupture of larger events is controlled by strong and highly stressed areas within the volume ΔV , the recurrence time $\bar{t}$ should then be calculated on the basis of frequency - magnitude data selected from these areas only, as determined by the contours of energy index or seismic stress and/or modelled stress, instead of from the entire volume.

The most significant deviations of observations from the frequency-magnitude relation are those at the largest observed magnitudes, since they may influence the expected recurrence time for the maximum magnitude event. In general, recurrence times beyond the time span of the data set Δt should be treated with caution.

In addition, it is useful to know the distribution of actual recurrence times about the estimated mean. Given the mean, the type of distribution, the standard deviation of the observations and the time of occurrence of the last event, either cumulative probability (i.e. the probability that an event would already have happened) or future conditional probability may be estimated. Failing that, one can evaluate the best estimate of the empirical probability P_T that a given volume ΔV will produce an event of magnitude greater than m within a specific time T after the preceding event of this size. Given the latest n observed recurrence intervals $\bar{t}(\geq m)$, of which n_T are smaller than or equal to T ,

$$P_T = (n_T + 1) / (n + 2), \text{ with uncertainty } \delta P_T = \pm 2 \sqrt{P_T(1 - P_T) / (n + 3)}$$

For larger data sets, δP_T approximates the 95% confidence interval. For example, consider a sequence of the last 15 recurrence intervals in days, for events with $m \geq 3.0$ in one of the mining areas in South Africa: 86, 33, 118, 58, 31, 107, 61, 77, 10, 17, 8, 13, 4, 26, 4. Then, the empirical probability that this area will produce an event with $m \geq 3.0$ within, say, $\bar{T} = 90$ days of the preceding one is $P_{90} = 0.82$ with

$\delta P_{90} = \pm 0.18$, which could be considered reasonably significant. For the same data set, $P_{30} = 0.47$ with $\delta P_{30} = \pm 0.23$ would not be considered significant.

In general the b -value is influenced by the following characteristics of the geomechanical system under consideration:

- * the stiffness, i.e. the ability to resist deformation with increasing stress
- * the level of stress
- * the rock mass heterogeneity.

The stiffer the system, the higher the b -value. This observation conforms with reported decreasing b -values with increasing stress, since there is a general loss of stiffness with increasing stress in a regime undergoing inelastic deformation. In the absence of significant tectonic stresses, intermediate and large seismic events usually occur after considerable mining has taken place in an area degrading the stiffness of the system.

The rock mass heterogeneity is defined by the spatial distribution of sizes and distances between strong and/or stressed and weak and/or destressed patches of rock where seismic sources may nucleate and be stopped. In general, for the same stiffness, an increase in rock heterogeneity results in a higher b -value, since it is more likely that an initiated rupture will be stopped by a soft or hard patch before growing into a larger event.

A rock mass subjected to mining is strongly influenced by excavations, pillars, induced fracture zones, and associated changes in stress. These induced heterogeneities influence the stiffness and the stress regimes of the system and thus the activity rate, size of the larger observed events and the b -value. For example, the introduction of strike stabilizing pillars in the West Rand Region in 1980 and, recently, the sequential grid layout, both reduced the size of the largest events experienced compared to those in traditional longwall mining. Figures 9.5.2 show cumulative apparent volume vs energy index, size and the time-of-day distributions derived from seismic data recorded during a shaft pillar extraction. Note the softening of the pillar during May 1993, manifested by persistent decrease in energy index EI associated with an increase in the rate of seismic deformation and associated changes in the size and time distributions.

Similarly, the d -value defining the slope of the $\log E$ vs $\log M_0$ straight line fit, called the E - M_0 relation, tends to correlate with the system stiffness - the stiffer the system the steeper the line and the higher the d -values - see Figure 9.5.3. The E - M_0 relation for the stiff system does not extend far into the large moment domain, since it does not produce large events until its stiffness is degraded and the d -value drops. For a given slope, an increase in the c -value of the E - M_0 relation reflects an increase in stress - the apparent stress of a typical event with $M_0 = 1 \text{ Nm}$, or $m = -6$, would be $\sigma_A(M_0=1) = \text{rigidity} * 10^c$.

The observed data sets of mining-related seismicity frequently exhibit fairly complex behaviour. The frequency-magnitude distributions of seismic events associated directly with tunnel development or stoping and those related to geological features of different sizes may have different forms, and when superimposed may affect the estimated recurrence times of larger events - see Figure 9.5.3. In addition, the relatively subjective choice of volume(s) of interest, the range of spatial and the degree of temporal correlation of seismic activity and the availability and quality of data may also influence the results. Therefore many data sets could be considered anomalous, having either a peculiar b -value, i.e. outside $0.5 < b < 1.5$, and/or strongly deviating from the Gutenberg-Richter model. In such cases it is important to determine the physical factors affecting the distribution and to ensure that the interpretation is offered in the context of the specifics of the data set.

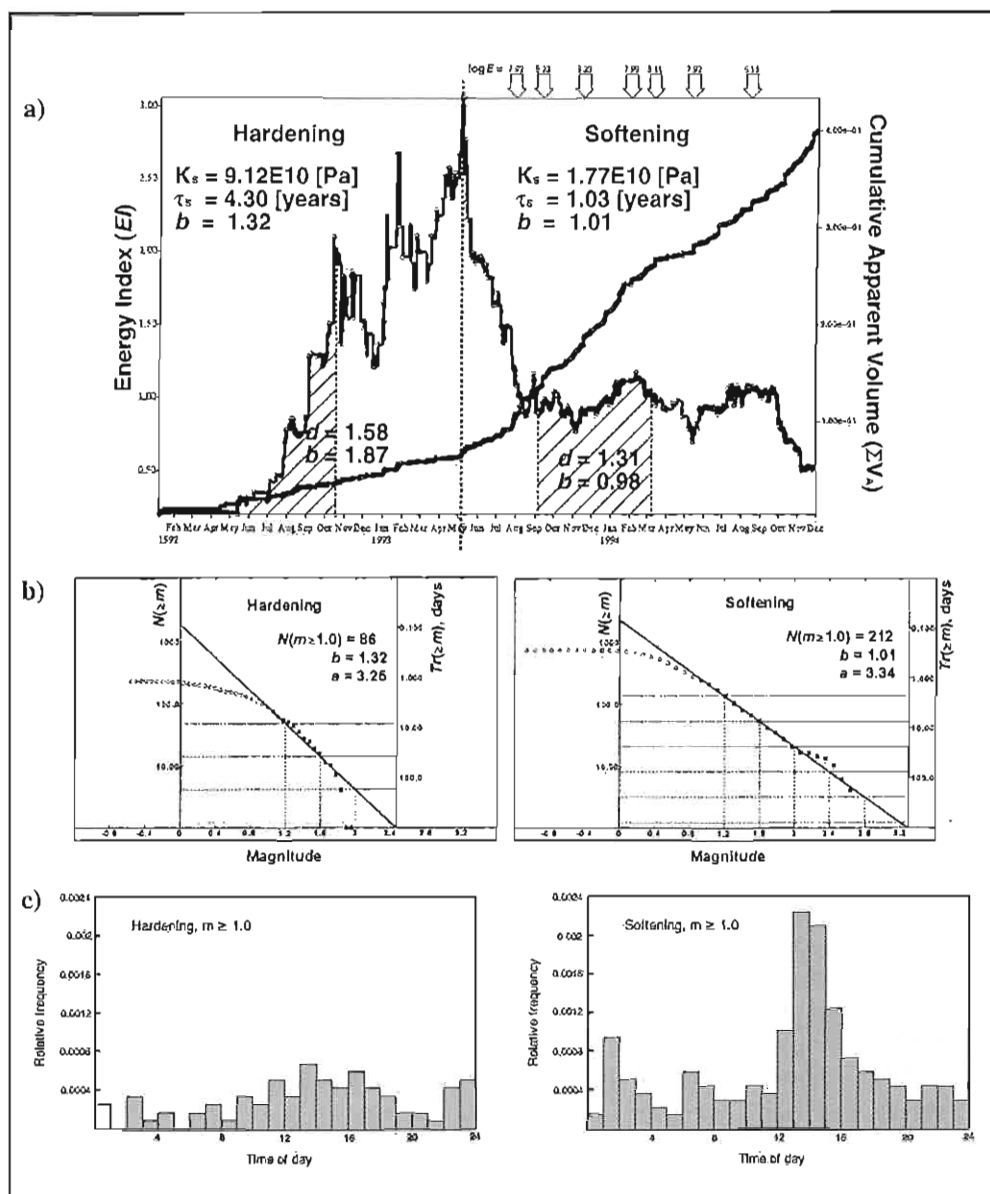


Figure 9.5.2(a) Time history of cumulative apparent volume ΣV_A , and energy index EI , for the seismicity associated with shaft pillar mining within the volume $\Delta V = 51.2 \cdot 10^6 \text{ m}^3$. Note that all large seismic events, $\log E > 7.5$, occurred after the pillar had lost the ability to maintain stress. Note also the differences in d -value (slope of the $E-M_0$ relation) and in the b -value between two six-month periods, indicated by shading, of similar average energy index EI but considerable difference in stiffness.

Figure 9.5.2(b) The cumulative frequency-magnitude distributions for the period January '92 to May '93 (hardening); and June '93 to December '94 (softening). Note the characteristic misfit (the lack of larger events) between the data and the model during the overall hardening regime, and the incidence of larger events during the softening period.

Figure 9.5.2(c) The time-of-day distributions of seismic events with $m \geq 1.0$ for the hardening and softening periods; note that hazard for $m \geq 1.0$ at 23h00 until 01h00 and at 10h00 and 12h00 is slightly higher during hardening.