

prestressing support units therefore appears to be beneficial in many circumstances, and needs to be evaluated in detail.

The second issue concerning support spacing in certain ground control districts in shallow mines is the prevention of large blocks sliding out on inclined planes, resulting in prop type support units being pushed out and failing in toppling mode. A simple method to reduce the possibility of such a failure is to ensure that at least three support units will support an unstable block, and that the centre of gravity of the block lies within the area defined by the support units. However roofbolting, designed with adequate pretensioning and shear resistance, is the more conventional way of ensuring stability in such situations.

In **deep mines** where the hangingwall is intensely fractured and may be subjected to seismic deformations, it might appear that the need for better areal coverage or support interaction would be significantly greater than for shallower situations. However, the clamping forces to the predominantly sub-vertical slabs running parallel to the stope face in the hangingwall, provided by the dilation of the fracture zone ahead of the face, mitigates this to a considerable extent. The problem therefore is similar to the shallow situation in that blocks of unfavourable geometry are of main concern. The difference is that the size of the critical blocks will, in general, be smaller and their occurrence will be more frequent. Also under seismic deformation and subsequent differential decelerations across the spans between support units, the stability situation is much more complex. Thus superior area coverage is needed in typical deep mining situations.

It is clear, therefore, that **to improve safety, and to reap the immense economic benefits of reducing fallout dilution and stoping widths, improved support interaction and areal coverage specifications designed for specific geotechnical conditions are required.** Such improvements can most readily be accomplished by considering higher-density lower-force support systems, and systems utilizing headboards or other load-spreading configurations. [As an example, a strong tape mesh attached to a framework and suitable for use with prop-based support systems to provide almost 100% areal coverage for rockfall conditions has recently been developed. This system is unproven, but is currently undergoing field trials and a positive evaluation is anticipated.]

4.3.3 Support Resistance And Support Spacing

Support resistance is the concept whereby the force generated by a support unit or the smallest representative section of a support system is averaged over the tributary area of hangingwall to be supported by that unit or portion of a system. Support resistance is expressed in kN/m^2 and is used in the setting of design criteria and the evaluation of support systems for rockfall conditions. The force generated by a support unit at a particular distance behind the stope face is determined from the force:deformation curve appropriate to the rate of closure and the amount of compression (closure) experienced by that unit in its current position. Assuming that the closure rate for the situation illustrated in Figure 4.3.1 is 10 mm/day or 20 mm/m of face advance (corresponding to a two day cycle), then the force generated by the units in the area of interest is gained from Figure 4.3.2.

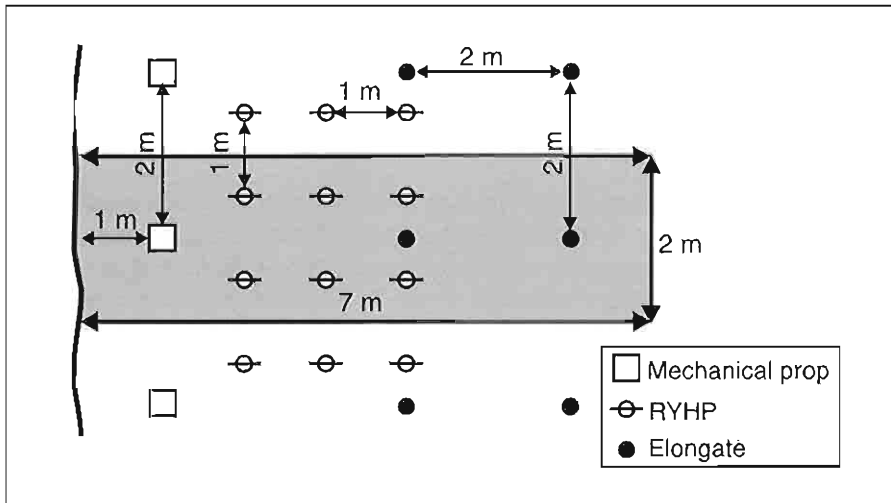


Figure 4.3.1 'Tributary area' segment of a support system, used for calculation of support resistance

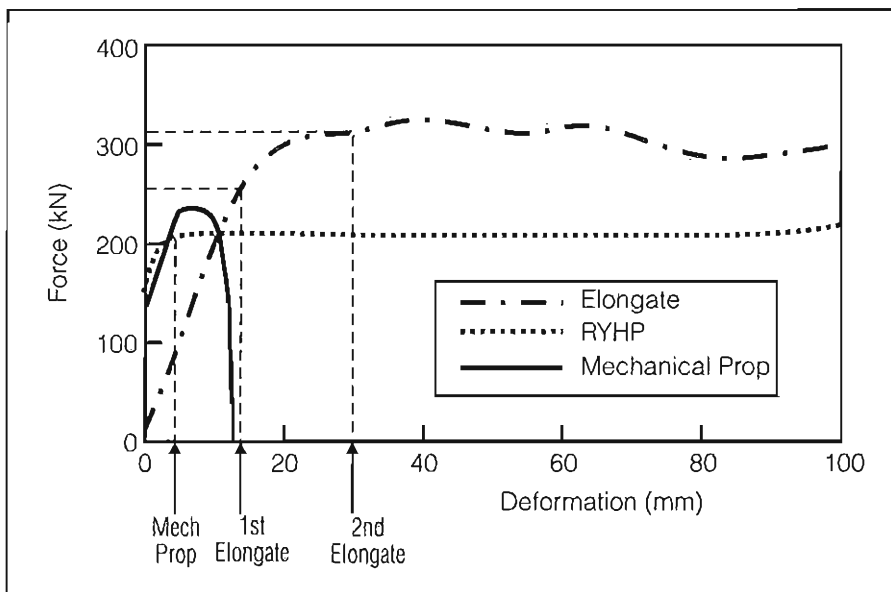


Figure 4.3.2 Forces generated by individual units for calculation of support resistance

SUPPORT RESISTANCE CALCULATION:

Mechanical prop - installed that day; prestressed; closure 3 mm

6 x RYHP at slow yield force of 200 kN

First row elongate - installed previous day; closure 13 mm

Second row elongate - installed for 3 days; closure 30 mm

FORCE

200 kN

1 200 kN

250 kN

310 kN

1 960 kN

Tributary area 2 m x 7 m = 14 m²

Support resistance = 1960 / 14 = 140 kN/m²

The criterion for rockfalls is the deadweight of rock that could potentially fall out per m^2 of hangingwall. In effect this is pgh , where p is the density of the rock and h is the potential thickness of fall-out. The SDA program quantifies the support resistance for each row of support and compares the result to the criterion; the procedures are discussed more fully in section 4.4.

The support resistance concept is a first step in the design process and ensures that the support elements, taking into account their yield characteristics, are never overloaded as a whole, and are always capable of carrying the deadweight of potentially loose rock in the area where required to do so. In effect, tributary area is an oversimplification of the situation in a highly discontinuous rock mass. Support forces are not evenly distributed over the tributary area but are less concentrated in the zones between the support elements depending on the specific design, and on the orientation and spacing of fractures and joints. The occurrence of fallouts between support units is evidence of this. Careful consideration therefore needs to be given to the strike and dip spacing of support units, with the objective of ensuring adequate overlap of their interaction with the rock mass.

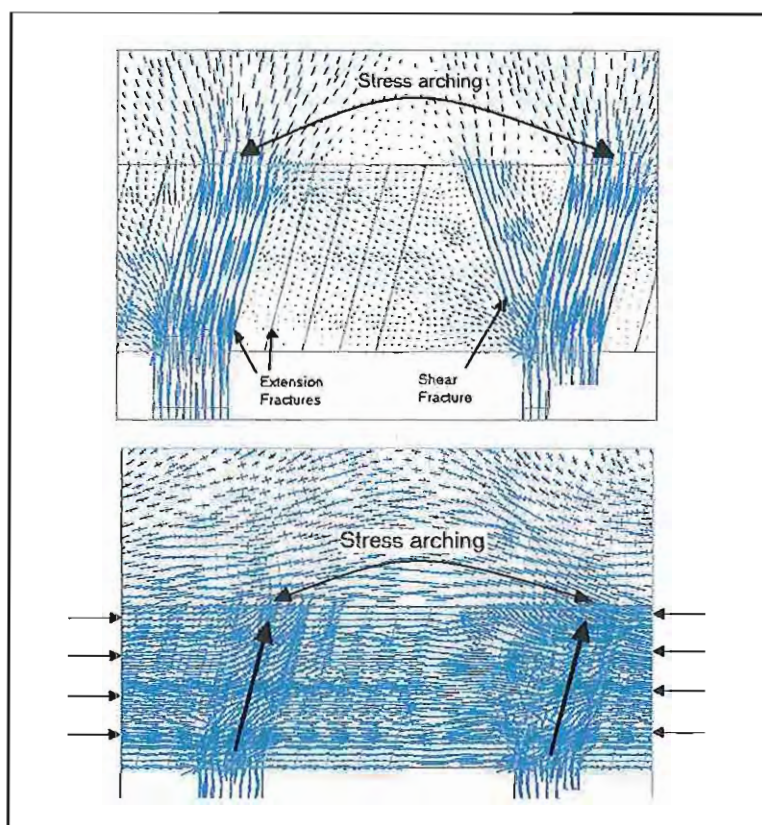


Figure 4.3.3 Numerically modelled force trajectories generated by support units. Upper diagram: no clamping stresses; lower diagram: 0.5 MPa horizontal stress present. When horizontal stresses are present and face fractures dip steeply, hangingwall stability is enhanced.

Currently, support spacing is based on local knowledge and the requirements to meet the overall support resistance criterion. In general, because stress fractures are parallel to the stope face, the requirement should be to have the strike spacing less than the dip spacing. Unfortunately, all too often the strike spacing is dictated by the need to scrape between rows of support. When this occurs to the extent that it appears that the interaction between units is sub-marginal, steps should be taken to spread the load by the use of headboards or other devices. [Research into this problem is ongoing and clearer guidelines should become available shortly. Figure 4.3.3 is a numerical simulation of how support forces may be distributed in a regularly fractured hangingwall bounded by a weak bedding plane one metre up. It is seen that the support forces are strongly channelled into the fracture-bounded slabs immediately above the support units, while the stability of the intervening rock is barely affected. However, in the lower diagram where dilatationary horizontal clamping stresses are present, support forces are transmitted across the fractures and into the hangingwall beam. Preliminary calculations show that stability is considerably enhanced when the dip of the face fractures is steep, but that there is a negative influence of combinations of shallow dipping fractures.]

4.3.4 Energy Absorption

The energy absorption concept is based on the principle that, during dynamic deformation of the hangingwall, kinetic energy is imparted to an ejected block which must be absorbed by the support units without undergoing undue deformation: $E_k = \frac{1}{2} mv^2$, where m is the mass of the rock loading the support units (again tributary area is assumed), and v is the peak velocity to which the rock is accelerated by the seismic event. At present, the value given to this critical parameter is 3 m/s - c.f. discussion in Chapter 1.4.4.

During deformation, the support units must also absorb potential energy given by mgh , where h is the dynamic closure. The E_k energy therefore corresponds to the *extra* component of support resistance required to absorb the kinetic energy of the ejected block - see figure 4.4.13. These issues are discussed in section 4.4 and, again, the SDA program (section 4.4) is available to help carry out the necessary calculations.

4.4 DESIGN AND EVALUATION OF SUPPORT SYSTEMS

In this section, currently accepted procedures for setting stope support design criteria are introduced, and methodologies are suggested which allow support systems to be designed and evaluated against these criteria.

4.4.1 Setting Of Design Criteria

The first step in support design is to set criteria against which the support system must be evaluated. The initial decision is whether the support has to cater for rockfalls only, or rockbursts and rockfalls. Then the height of potential fallout needs to be determined. This involves careful analysis of the near hangingwall stratigraphy, geologi-

cal structures aid stress fracturing to determine potential release surfaces and the location of the highest of these relative to the slope. Importantly, this needs to be corroborated by investigations of underground collapses from which all relevant data should be stored in a database. This can then be interrogated and the height of falls plotted in cumulative percentage format. These data need to be examined to establish if they comprise more than one population. For example, the larger falls of ground could be associated with faults, indicating a higher value for the criterion in the vicinity of faults but conversely a lower value for normal conditions. By engineering judgement of all these data, the height of potential unstable strata should be established and the support resistance or energy absorption criteria calculated. These procedures, suitable for 'rockfall' and 'rockburst' support designs, are dealt with below.

For pure **rockfall** control, the **support resistance** of the support system has to be sufficient to carry the deadweight of the strata involved in a potential fall:

$$SR \geq (\rho t_{ff}) g \quad [N/m^2]$$

where ρ is the density of the hangingwall rock (2750 kg/m³ for quartzite), $g = 9.8$ m/s², and t_{ff} is the thickness (m) of the potential fall. This approach is relatively conservative, in the sense that cohesive and frictional forces on the surfaces bounding the potential fall are deemed negligible and are ignored.

The thickness parameter t_{ff} needs to be set for each ground control district under consideration. It is best defined as the *distance to the furthest known unstable parting plane in the hangingwall*. Analysis of a (sufficiently large) set of current actual rockfall thicknesses can give useful information here - Figure 4.4.1.

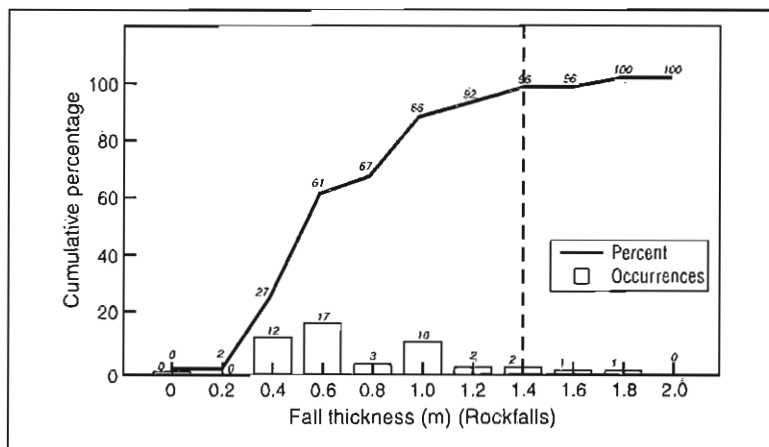


Figure 4.4.1 Cumulative rockfall thicknesses for a particular ground control district

Judgement needs to be exercised as to which percentile on such a cumulative fallout thickness graph should be used, and the cases comprising the top 10% should be examined. If the majority of these heavy falls occur in a particular area, then this could be delineated as a separate ground control district with its own more stringent support standards. Alternatively, if they are associated mostly with identifiable

features such as faults or rolls, then this could be catered for under the mine standards, where appropriate local increases in support resistance would be specified. If in the example of Figure 4.4.1 it were accepted that the *standard* support system should prevent 95% of all observed fallouts, the thickness of strata to be supported would be $t_{cf} = 1.4$ m, and the corresponding support resistance criterion would be $2750 \times 1.4 \times 9.8 = 38 \text{ kN/m}^2$.

Two important issues need to be borne in mind with this approach:

- (i) A database of fallout thicknesses reflects actual falls of ground which took place in the past, but obviously *in the presence of support installed at the time*.
- (ii) Unless the database includes at least about *100 instances of significant falls* (unlikely in the case of small, or recently exploited, ground control districts), the critical upper tail of the fallout distribution may be missed or greatly under-estimated. For these reasons due caution needs to be exercised, and consideration given to the use of appropriate factors of safety (section 4.4.5), dependent on an estimate of the relative *variability* of geological and other weaknesses in the hangingwall. For example, if large falls associated with difficult to recognise hazards (such as 'domes' on the platinum mines) are thought to be possible, a particularly conservative approach is indicated.

In the case of **rockburst** prone areas, the available support resistance must not only be able to carry the deadweight of a potentially ejected block, it must be able to absorb its **kinetic energy** E_k . The requirement is that the support has to bring to rest, within a specified distance, a detached block of thickness t_{rb} and mass $m = \rho g t_{rb}$ in the hangingwall, which has been accelerated to a peak velocity v by the strong ground motion from a seismic event - Figure 4.4.2. The kinetic energy involved is given by

$$E_k = \frac{1}{2} m v^2 \quad (\text{J/m}^2).$$

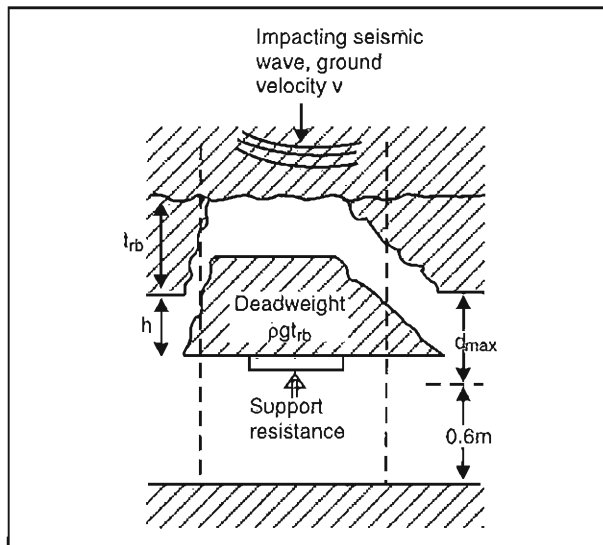


Figure 4.4.2 The rôle of support in rockburst conditions

The controlling parameter - the likely maximum thickness t_{rb} of an ejected block - can be defined as the distance to the highest (dynamically) unstable parting in the

hangingwall. Again, useful information can be gained from analysis of actual rockburst ejection thickness data - Figure 4.4.3.

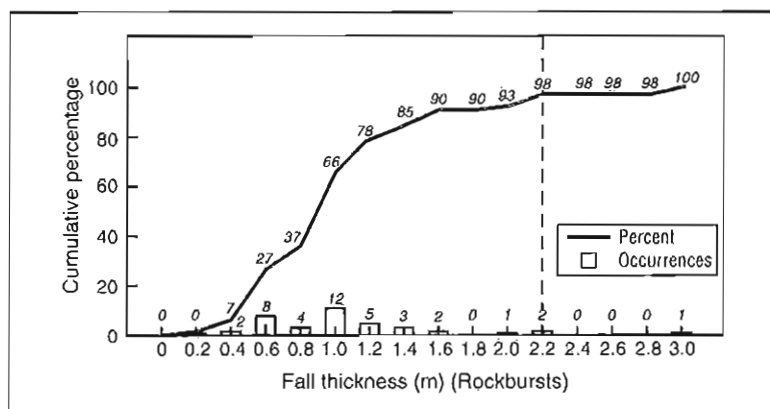


Figure 4.4.3 Cumulative percentage of ejected block thicknesses for Carbon Leader Reef.

From this figure the (overall) 95 % percentile thickness for the Carbon Leader Reef was 2.2 m, while the maximum observed thickness was 3.0 m. The actual Carbon Leader ejection thicknesses appropriate for a given ground control district will vary across the ore body, depending largely on the distance between the reef and the Green Bar. Again, due caution should be exercised, for similar reasons to those discussed under rockfall criteria above.

Assuming an estimated block ejection thickness of 2.2 m, i.e. a mass of 6050 kg/m², the minimum kinetic energy absorption requirement per square metre of stope hangingwall that a support system should be able to provide to stabilise the stope hangingwall, assuming an ejection velocity of 3 m/s, is $E_k = \frac{1}{2} 6050 \cdot 3^2 = 27000 \text{ J/m}^2 = 27 \text{ kJ/m}^2$. This is a representative E_k criterion for the 'average' Carbon Leader Reef.

A further important consideration in rockburst support system design is the *maximum deformation* $d_{\max}$ which a support unit can be allowed to sustain before bringing a rockburst-ejected block to a standstill. To avoid serious danger to life and limb, the open height in a stope after a rockburst should, if at all possible, not be permitted to fall below about 0.6 m. Thus $d_{\max}$ should not exceed $(S_w - 0.6)$, where S_w is the stope width at installation of the support unit concerned. Furthermore, after assumed arrest of the falling block, the (static) resistance of the support unit cannot possibly be less than the deadweight mg of the block, else further unlimited collapse would occur. Thus

$$d_{\max} = \text{Minimum of: } (S_w - 0.6); \text{ or } d \text{ such that } SR_{\text{static}}(d) = mg.$$

Finally, the acceptable **spacing** of support units and areal coverage requirements need to be estimated, as discussed in section 4.3.2.

It is necessary that these procedures be carried out for each geotechnical region. This highlights the need of not relying too heavily on accident data, as for each region the quantity of data available is likely to be sparse.

4.4.2 Establishment Of Closure Rates For Ground Control Districts

Stope closure rate is an important input parameter in the support design procedure (sections 4.4.3/4) and is a useful and simple measurement indicating rock mass behaviour. The rate and amount of closure after mining is dependent on several factors, most of which are included as parameters in the delineation of ground control regions. It is likely that closure rate will vary from region to region and therefore these rates need to be established.

Where the rock mass is fractured and deforming inelastically, it is not currently possible to model the rate of closure realistically because of the many and varying factors involved. It is thus necessary to measure these rates underground for each region. The detail of measurements can be considered as an evolutionary process with the end objective being a well-populated database of measurements.

If no actual closure measurements are available, quick estimates can be made by visiting each region and assessing the compression of support units at various distances behind the face. Dividing by distance to the face will give closure rates in mm/m of face advance, or by examining mine plans to determine how long it took to mine that distance would give closures in mm/day. To validate these estimates, a few simple closure meters should be installed in each region. Initially these need only be read monthly, for example when production measuring is carried out, and the data analysed.

As time passes or resources are made available, more detailed and sophisticated measurements should be taken (Chapter 10.3). In particular, if rates of face advance vary within a region, the effect of this on closure rate needs to be determined. Continuous recording of closure will help elucidate the time-dependent component.

Where roofbolting is used, the dilation in the roofbolted area needs to be determined to assess whether yielding tendons are required. This can be done by installing extensometers into short holes drilled into the hangingwall.

4.4.3 Stope Support Design Methodology For Rockfall Conditions

The proposed design methodology compares the actual support resistance of the system to the criterion at any distance from the stope face, taking into account the effects of stope closure. The methodology thus ensures that both the face area and permanent support within the working area meet the criterion. It also caters for support systems comprising several types of support units, including tendons used to support the hangingwall in suspension mode. Using the methodology, different support systems can be evaluated, and the most cost-effective one chosen which both satisfies the support resistance (safety) criterion and best integrates with the chosen mining method.

Any support units installed in a stope are immediately acted upon by stope closure. Depending on the force-deformation characteristics of the support unit, this closure could either degrade or increase its ability to generate load. In order to take this into account, it is necessary to represent a support system as a support resistance-deformation curve which is a function of stope closure and thus of the distance behind the stope face.

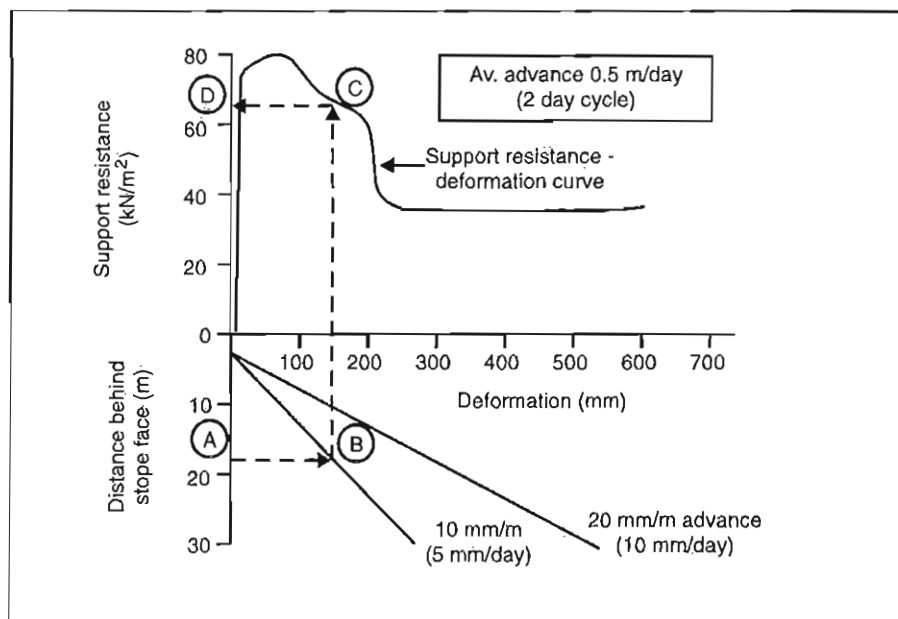


Figure 4.4.4 A support resistance-deformation design chart, which allows stope closure and distance behind the stope face to be taken into account

Figure 4.4.4 is a graphical representation of the design method. The upper section is the support resistance-deformation graph for the proposed system being evaluated. The lower section is a nomogram that converts distance behind the face to compression of the support units for various rates of *closure* expressed in mm/m (mm of closure per metre of face advance); or equivalently, if the face advance rate is specified, in terms of mm/day. The method also takes into account the distance behind the face that the support is installed by starting the closure rate lines on the y-axis at this distance.

The graphs can be used to determine the support resistance for any distance behind the stope face and for any closure rate. In the example, the support is assumed to have been initially installed 3 m from the stope face and the stoping is on a 2-day cycle. Considering point A some distance behind the stope face, a horizontal line is traced until it intersects the line representing the stope closure rate at point B. From B a vertical line is traced to intersect the support resistance-deformation curve of the support system at point C. From C a horizontal line is traced back to the y-axis at D, where the support resistance of the support system at that particular distance from the face for the specific closure rate can be read off.

The graphical representation may be further extended by separating the face and permanent support areas, and plotting the support resistance-deformation curves of the support systems used in both these areas. This is shown in Figure 4.4.5, where a mine pole system and a mechanical prop support system are shown in the face area for comparison. In the permanent support area, a timber pack system is compared with a yielding timber prop support system. This allows the support resistance of a specific support system to be tested against the support resistance requirement of a particular ground control district as determined in section 4.4.1 (38 kN/m^2 in the example).

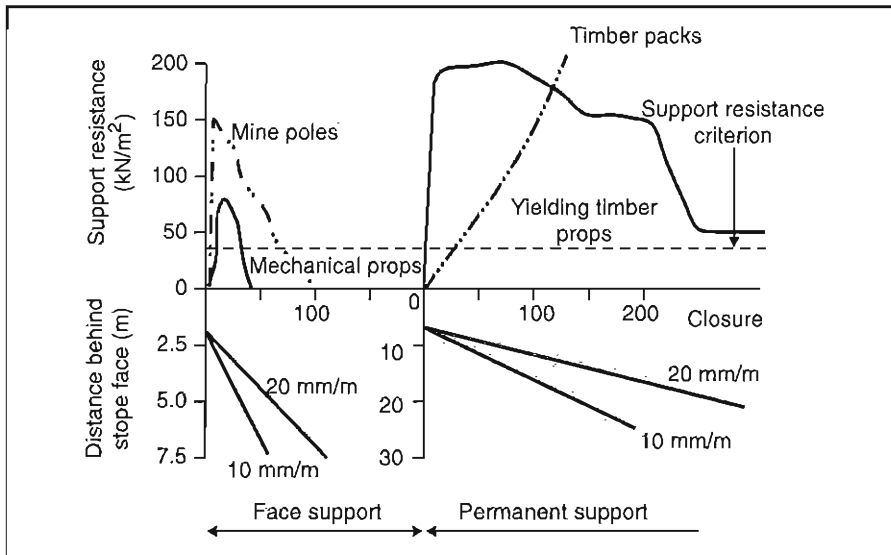


Figure 4.4.5 The support resistance-deformation curves of support systems for both the face and permanent support areas

It is significant that some of the support systems in Figure 4.4.5 fail to meet the criterion of 38 kN/m^2 under certain conditions. For example, for a closure rate of 20 mm/m (10 mm/day , assuming a 2-day cycle), the mine pole support system will fail to meet the criterion once a row of the support units comprising the system is 4 m or more behind the stope face. For a closure rate of 10 mm/m (5 mm/day on a 2-day cycle), the equivalent distance is 6.5 m . The mechanical prop support system will fail to meet this criterion, and the props would suffer damage, at 3 m and 4.5 m behind the stope for closure rates of 20 mm/m and 10 mm/m respectively. The pack system also appears not to meet the criterion in the working area where the closure rate is 10 mm/m . However, a small additional amount of compression of the packs, due to the weight of a potential fall of ground, would (probably safely) generate sufficient resistance in the packs to carry the extra load.

A computer program, **SDA**, is available which automatically performs all the steps in this design evaluation process. At present the program accesses a rather limited database of support unit load/deformation curves, but properly determined performance curves of units not in the database can easily be added. [The program is currently being enhanced to cater for more complex support systems such as those discussed below.]

Two different support element types used in conjunction can be evaluated by a slight extension of the methodology described so far. An example is a combination of hydraulic and profile props, where there is overlap of the two support types. Of importance here is determining the combined support resistance of the two support types for a given distance behind the stope face. The support resistance-deformation curves are plotted in the normal manner, but a closure rate curve is required for each support type as they are installed at different distances from the stope face. This is shown in Figure 4.4.6 where, by adding the two individual support resistances determined as in (C) and (D), the total support resistance of the system can be determined.

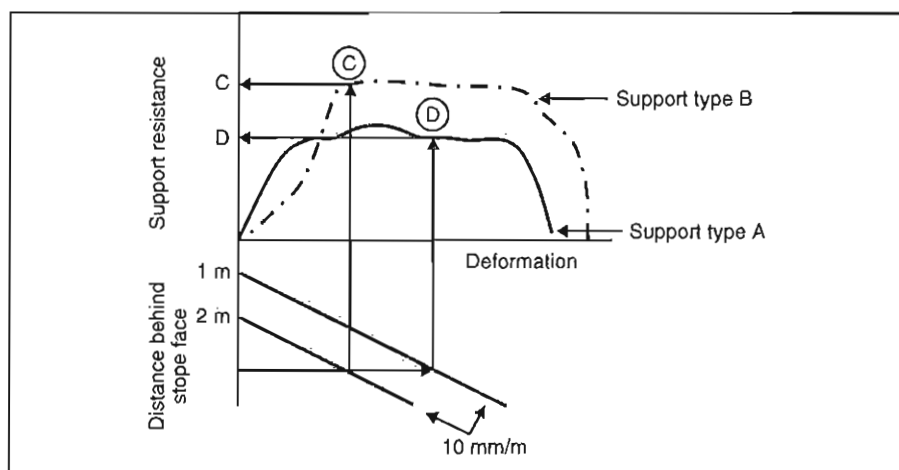


Figure 4.4.6 Multiple support types installed at different distances from the face

If this process is undertaken a number of times for different distances behind the slope face, then the change in support resistance from the face area into the back area can be plotted (Figure 4.4.7), and compared with the required support resistance criterion.

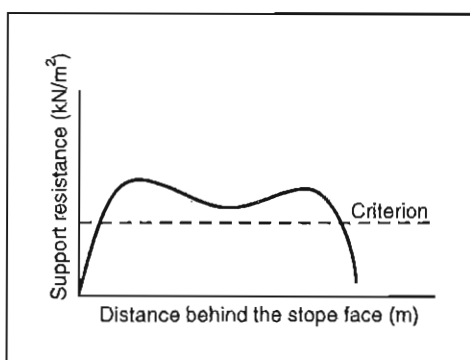


Figure 4.4.7

Total support resistance of multiple support types at various distances from the slope face

Support systems which incorporate rockbolts or tendons can be evaluated by assuming a constant support resistance contribution of such tendons, irrespective of slope closure. In order to ensure predictability and relative long-term stability of rockbolts, it is recommended that they not be loaded beyond their yield point, which is at about 70% of their ultimate strength - Figure 4.4.8. Thus the calculated support resistance should be based on 70% of the failure strength.

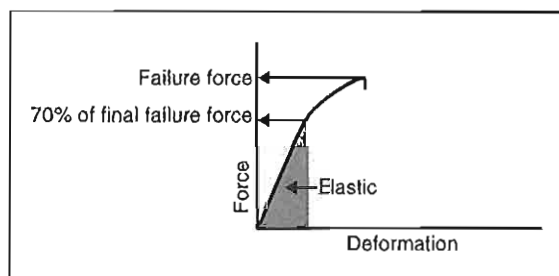


Figure 4.4.8

Force-deformation behaviour of a steel end-anchored rock-bolt

It is very important that tendons be anchored deeper in the hangingwall than the estimated t_{fr} fallout thickness; failing this the tendons cannot be assumed to contribute *at all* to support resistance requirements, but only to improving areal coverage and general hangingwall integrity.

A support system comprised *only* of rockbolts can be a viable solution (Figure 4.4.9), provided always that the following requirements are met: (i) the tendon spacing is acceptable in terms of local joint/fracture spacings and orientations, (ii) the anchors pin through the t_{fr} beam into the solid above, and (iii) the estimated support resistance exceeds the criterion.

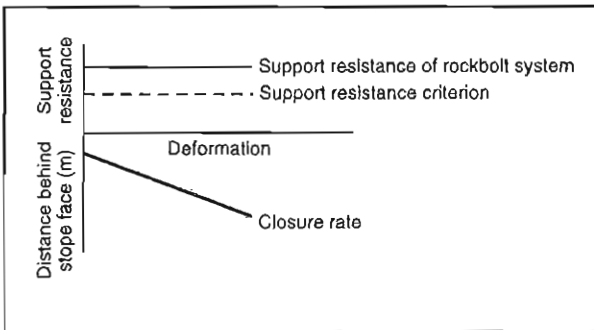


Figure 4.4.9

Application of the stope support design methodology to evaluate the support resistance of rockbolts

Alternatively, a *hybrid* system made up of rockbolts in conjunction with other support units can satisfy the support resistance criterion over the full working area - Figure 4.4.10. The methodology is illustrated in Figure 4.4.11, in which the contribution of the rockbolts is added to that of the props.

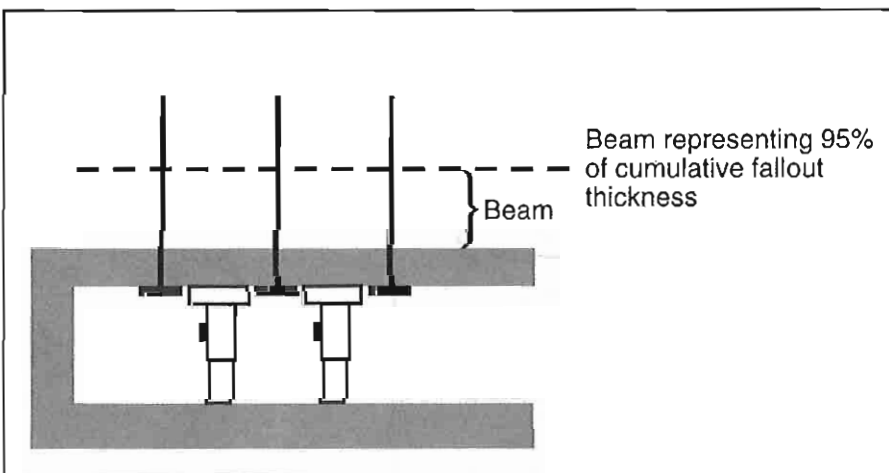


Figure 4.4.10 Hybrid support system comprised of rockbolts and props to meet support resistance requirements

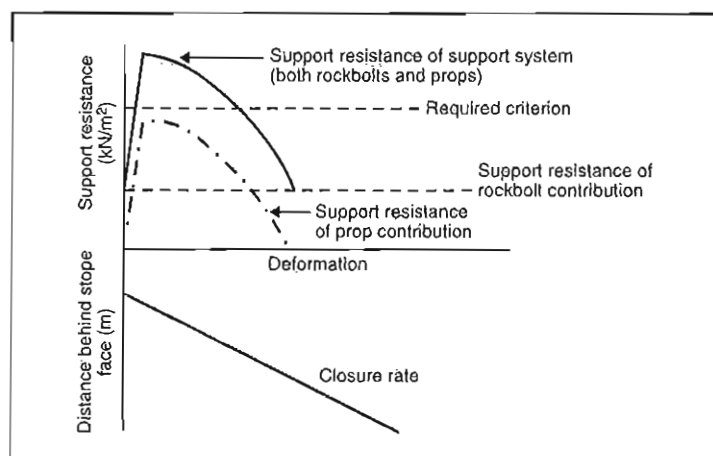


Figure 4.4.11
Linearly super-
imposed sup-
port resistance
of rockbolts
and props

4.4.4 Stope Support Design Methodology For Rockburst Conditions

The design/evaluation of a stope support system required to absorb energy with the objective of reducing rockburst damage depends on a number of variables, the most important of which is the ability of the support system to yield during *rapid deformation* and so absorb energy. Other variables which affect the ability of stope support systems to absorb energy have been discussed previously, and include the spacing of the support units, the expected velocity of dynamic stope closure, and the rate of normal stope closure which affects the support resistance and energy absorption capacity of support units as their distance from the face increases. Figure 4.4.12

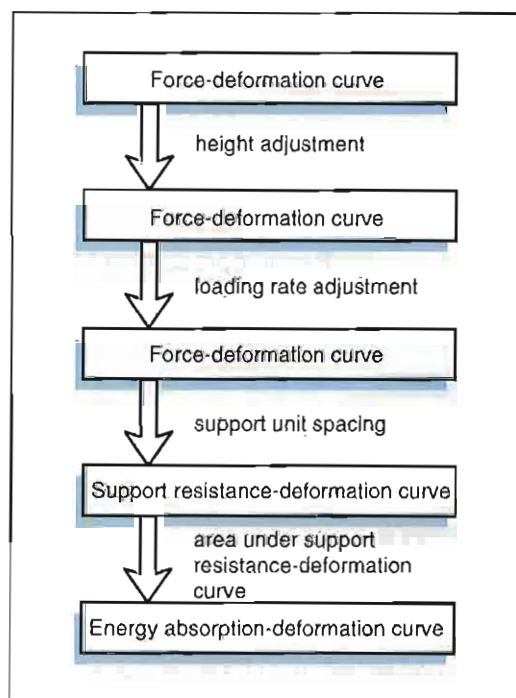


Figure 4.4.12
Process for estimating dynamic
support properties

gives a flowchart of the processes involved in estimating the *dynamic-loading* support resistance and kinetic energy absorption curves of a given support system. Figure 4.4.13 illustrates how the kinetic energy absorption capacity of a support system is derived (*excess* area under the dynamic support resistance curve, calculated from any prior stope deformation d up to limit defined by $d_{\max}$, c.f. section 4.4.1 and Figure 4.4.13). Figure 4.4.14 illustrates typical energy capacity curves for a number of common support systems, and shows how these capacities diminish as prior deformation (due to normal stope closure) increases.

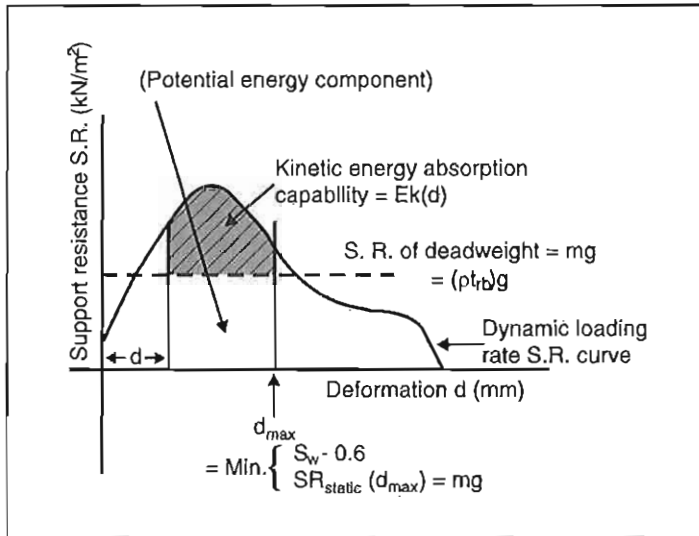


Figure 4.4.13 Derivation of energy absorption capacity of given support system

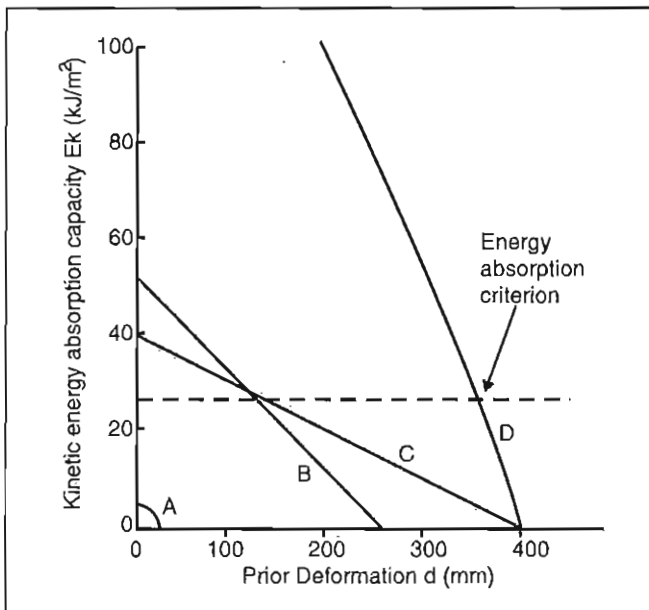


Figure 4.4.14 Energy capacity curves for A: mine pole (2 m² spacing), B: hydraulic prop (1.5 m² spacing), C: reinforced foamed concrete pack (6.8 m² spacing), D: solid mat pack (10 m² spacing) support systems.

Most support units and support systems have a finite amount of energy that can be absorbed during deformation, the main exception being backfill. Based on laboratory tests, pack support systems might appear to have a continually increasing support resistance and unbounded energy absorption capacity with increasing deformation. However, it is estimated that the useful energy absorption ability of pack support systems is limited to about 50% compression, since most pack types clearly do lose structural integrity after large deformations in underground conditions, particularly under dynamic closure. These considerations affect the determination of the maximum deformation $d_{\max}$ parameter (defined in section 4.4.1), which plays an important role in the calculations illustrated in Figure 4.4.13.

Once energy capacity curves similar to those in Figure 4.4.14 are available, a process similar to that introduced for rockfall design (section 4.4.3) can be followed, except that now behaviour is tested against the kinetic energy criterion E_k defined in section 4.4.1. An illustration is given in Figure 4.4.15.

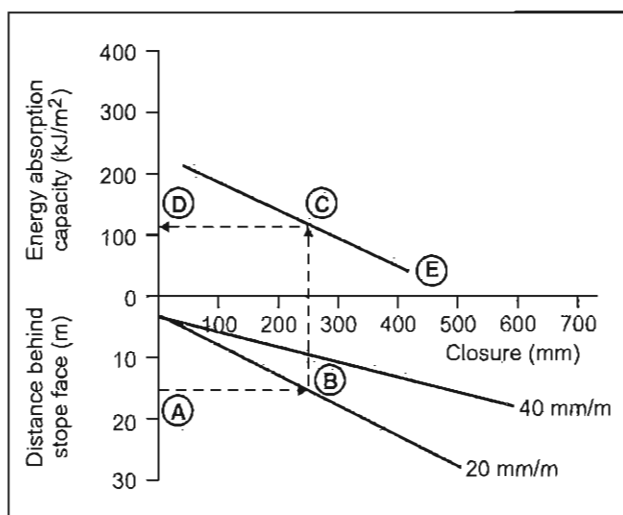


Figure 4.4.15

An energy capacity-deformation design chart, which allows stope closure and distance behind the stope face to be taken into account

Figure 4.4.15 combines the graph of energy capacity with a nomogram in the lower half of the diagram which relates the amount of stope closure experienced by a support unit installed a specific distance behind the stope face for various rates of closure. In this example curves of closure rate expressed in mm/m of face advance are used. The negative y-axis is again distance behind the stope face, and the closure rate lines intersect the y-axis at the distance behind the stope face at which the support is installed (3 m in this example).

Figure 4.4.15 illustrates how the graphical representation may be used. Consider some distance behind the stope face, point A. A horizontal line is traced from A until it intersects the line representing the stope closure rate, point B. From B a vertical line is traced to intersect the energy-deformation curve of the support system, point C. From C a horizontal line is traced back to the y-axis at D where the available energy can be read off and compared with the kinetic energy criterion E_k : 27 kJ/m² in this example. Only at point E (corresponding to about 25 m behind the face) does this particular support system fail to absorb the energy of a 2.2 m thick, 3 m/s ejected block, while still retaining sufficient support resistance to carry the deadweight of the block.

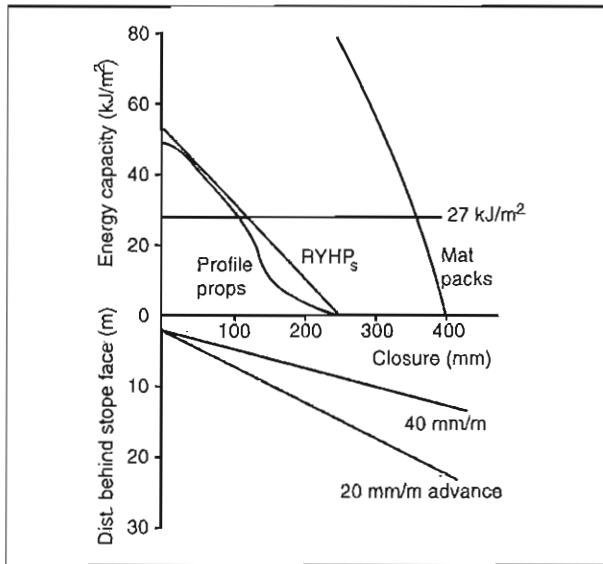


Figure 4.4.16
Rockburst design charts
for various support sys-
tems

Figure 4.4.16 shows some typical support systems made up of timber packs, hydraulic props and profile props plotted in the way described above. This graphical representation may be further extended by separating the face and permanent support areas and plotting the energy-deformation curves of the support systems used in both the face and permanent support areas. This is shown in Figure 4.4.17 where a mine pole support system and a hydraulic prop support system are shown in the face area for comparison. In the permanent support area a timber pack support system is compared with a profile support system and with an RSS grout (foamed concrete) pack support system.

Again, the **SDA** computer program is available to assist in this design evaluation process.

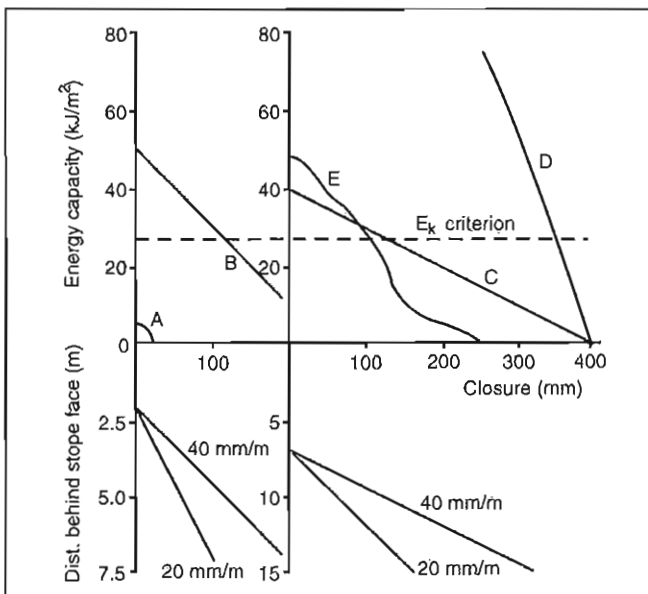


Figure 4.4.17
Typical design
charts for face
and back areas.
A: mine poles,
B: RYHPs,
C: reinforced
foamed concrete
packs,
D: mat packs,
E: profile props

Below are characteristics of units constituting common support systems, determined on the basis of typical support spacings used in the industry for rockburst conditions.

Table 4.4.1 Examples of support unit characteristics under dynamic conditions

Support Unit	Max.Force	Yield Range	Spacing
Mine Pole	600 kN	0.08 m	2 m ² /unit
Mech. Prop	150 kN	0.05 m	2 m ² /unit
Yield.Mech.Prop	120 kN	0.25 m	2 m ² /unit
Hydraulic Prop	400 kN	0.25 m	1.5 m ² /unit
Profile Prop	500 kN	0.25 m	2 m ² /unit
Pack	4000 kN	0.50 m	10 m ² /unit
Reinforced foamed concrete pack	1000 kN	0.50 m	10 m ² /unit

Two different support element types used in conjunction can, as in the rockfall design procedures, be handled by first adding the individual support resistance characteristics and then evaluating the resulting areas representing energy absorption capacities.

Design of Tendon Based Systems for Rockburst Conditions. Where tendon support is designed into a support system with the purpose of absorbing part or all of the energy imparted to the rock, only yielding tendons are appropriate support units. In the design, the influence of possible shearing on bedding planes in the stope hangingwall is ignored and it is assumed that the energy absorbing capabilities of the tendons are only utilised once a seismic event occurs. However, where vertical dilation (bed separation) of the hangingwall beam to be supported is suspected, the amount of dilation can be determined by installing extensometers in the hangingwall (see Chapter 10.3). Corrections can be made for the dilation if the magnitude is sufficient to warrant the modification. As an example of energy absorbing capabilities of tendons, consider the force-deformation characteristics of a cone bolt, which is designed to yield during dynamic loading. The area under the force-deformation curve from dynamic laboratory tests quantifies the amount of energy absorbed by a cone bolt during dynamic loading. Similar to rockfall support design, it is imperative that these tendons be anchored at a point deeper than the estimated t_b thickness, and if the energy absorption capacity is inadequate, either reduced spacings or use of additional conventional supports needs to be investigated - Figure 4.4.18. To determine the energy absorption capability of a hybrid support system, the individual energy capabilities of the two systems are again *added*.

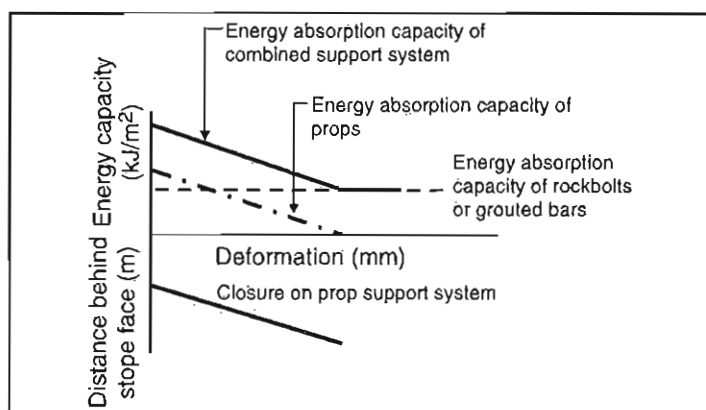


Figure 4.4.18
Superimposed energy capacities of yielding tendons and props

4.4.5 Cost Analysis, Risk Analysis And Factor Of Safety Evaluation

Having evaluated several possible support systems for a particular ground control district using the SDA program, it may be found that a number satisfy or exceed the design criteria. An important issue in deciding which of the options should finally be selected for implementation is the **cost** factor.

In assessing the total cost of a support system the initial material cost of the units may be only a small proportion of the installed cost. Other factors including transport costs from stores to stope, infrastructure requirements and their running costs, the cost of accessory equipment required for installation and its maintenance, installation costs and the cost of damaged or lost units, have to be taken into account and added to the large labour and supervisory costs associated with each of these operations.

It is obvious that reusable units have a huge advantage when considering initial and transport costs. In a well run RYHP system the expected life of a unit is seven years, i.e. it is used hundreds of times and transport costs are negligible. However, the infrastructure and supervisory costs required to keep such a system efficient are relatively high.

It is therefore necessary to carry out a thorough cost analysis to determine which of the proposed systems is most effective. It may even be found that employing a contractor to run the whole support operation or segments of it is an attractive option.

Having selected a particular support option, a formal **risk assessment** should be carried out. This should involve all aspects of the system starting at the risk of non-guaranteed supply of the units through to the risk of the system not providing the required support performance. The risk assessment carried out by the **supplier or manufacturer** of the components should be examined. The on-mine assessment should concentrate on the safety risks involved in the installation and removal (where appropriate) of the units, the likelihood of the units being blasted out, or for whatever other reason not providing the expected support performance, for example poor installation, buckling of elongates etc. Table 4.4.2 lists the number of fatal accidents associated with the installation and removal of the common types of units for the period 1990 - 1997. Note that it was not possible to normalise these data by the number of units installed. However, these fatalities comprise 12% of the total fatal accidents in stopes. This is a significant proportion, indicating less than adequate training and supervision in the routine, but hazardous, operations of support installation/removal.

Table 4.4.2 FOG fatalities associated with the installation (I) or removal (R) of support units

Support Elements	1990 I R	1991 I R	1992 I R	1993 I R	1994 I R	1995 I R	1996 I R	1997 I R	Total I R
Hydraulic	1 1	0 4	0 0	3 3	2 0	1 3	3 2	0 3	10 16
Mech.props	2 0	4 1	9 0	0 2	4 1	3 1	1 1	1 0	24 6
Packs	2 0	10 0	4 0	4 0	2 0	2 0	5 0	0 0	29 0
Sticks	7 1	3 1	5 0	6 0	4 1	2 0	3 0	1 0	31 3
Backfill	0 0	1 0	0 0	0 0	0 0	0 0	0 0	0 0	1 0
Total	12 2	18 6	18 0	13 5	12 2	8 4	12 3	2 3	95 25

In order to reduce the identified risks, cognisance should be taken of the results of the risk analysis in setting the installation **standards** and the **training** programme developed for the implementation of the system.

In addition, the inclusion of a **safety factor** in the design criterion needs to be considered. This should be based on relevant findings of the risk assessment and the uncertainties in estimating the original criterion. Also, the risk and effect of a support unit failing prematurely, being poorly installed or being left out of the pattern of support should be taken into account.

4.4.6 Field Trials

After the selection of the final candidate support system for implementation, it is advisable, particularly when a radically different support system is contemplated, that a field trial be carried out. The trial should take place in a two or three panel section over a period of about three or four months. Initially, while people are becoming accustomed to the new system, pressure to achieve high production rates should be relaxed and only during the final stages of the trial should maximum rates be striven for.

The objectives of the trial are to evaluate the effectiveness of the system in improving hangingwall stability and reducing dilution, safety aspects of the installation procedure, ergonomics, the refinement of standard procedures, the influence of the system on productivity, and the overall system cost-effectiveness.

The planning of the trial should involve the developers, manufacturers and suppliers of the components, to take advantage of their knowledge and experience of the system. During the trial, detailed monitoring of the system performance should be carried out and work study investigations undertaken. It is at this stage that valuable input can be gained from the mining crew and particularly from the people directly involved. Such input should be solicited by risk assessments during the trial and acted upon where appropriate. Suggested modifications to ergonomics and procedures can be expected.

The experience gained and modifications implemented during the trial would then be used in drawing up a training manual for mine-wide training necessary for implementation of the system, the refinement of standard procedures and the setting of quality assurance standards, not only for the support components but also their installation.

4.4.7 Implementation

The field trials described above, together with the training programme and documentation of standard procedures, should be seen as part of the implementation process. The stope and mining personnel used in the field trials can then be utilized as an underground training school for the benefit of other employees required to learn the system.

The implementation of a new support system should be phased in across the ground control district. Crews from candidate stopes having been through the training

course should spend several days in the training stopes to gain hands-on experience. Team leaders from the training stopes could spend some time with the novice crews in their stope to ensure the system is applied safely and efficiently and to assist in problem solving.

During the systematic implementation, monitoring should continue but in much less detail than in the field trial, again to assess safety and productivity benefits and cost effectiveness. Important at this time is to monitor and compare accident rates, stopping widths and face-advance rates before and after introduction of the system.

A valuable contribution to the industry as a whole can be made, following the successful implementation of a new support system, by writing up and publishing a comprehensive report on the exercise.

4.4.8 Backfill As Local Support

In South Africa, particularly in deep stopes, backfill is playing an important role as a *local support* medium, in addition to its potential contributions to regional support (Chapter 3.3.5) and to ventilation control. Numerous field studies have concluded that, where backfill of adequate quality is emplaced, hangingwall conditions and stability can improve dramatically; to the extent that the frequency and severity of rock-falls and rockburst damage are significantly reduced. Apart from these safety-related benefits, improvements in face advance rates and dilution control have been reported in a number of instances.

The positive effect of backfill on hangingwall conditions can be partially accounted for from the results of two detailed in situ rock engineering studies. The first study compared seismic ground motions in conventional vs backfilled panels. It was found that both the amplitude and duration of the dynamic deformations, as well as the peak ground velocity, were reduced significantly in the backfilled panels - leading to a reduction in susceptibility to rockburst damage. There are several possible reasons for this: damping of surface waves and consequent reduction of site-amplification effects, better transmission of seismic energy *through* the stope, and reduction of resonant unsupported hangingwall beam length.

The second study in a particular set of panels showed that, with backfill present, the measured horizontal stresses in the hangingwall near the face increased from low compressive or mainly tensile values (in conventional panels) to compressive stresses in both horizontal directions. The hangingwall beam is strengthened by these confining stresses and thereby made more intrinsically stable. [A known exception would be the case where the hangingwall is laminated, where backfill may over-generate horizontal stresses and thereby cause buckling failure of the strata; and the diametrically opposite strategy of 'cave' mining would be indicated - c.f. Chapter 3.5.5.]

Generally speaking therefore, the use of backfill together with good conventional face area prop support is a preferred medium of *local support* in deep mining, and in high stope width mining in general. An important proviso, however, is that, to realize the potential considerable benefits, the backfill has to be kept close to the face at all times (preferably < 6 m). Fairly stringent quality controls, particularly of densi-

ty and porosity which contribute to shrinkage control, has also to be exercised (Chapter 10.3.4).

Details of some of the aspects of backfill quality and placement technology were given in Chapter 3.3.5. As far as the choice of backfill type is concerned, an empirically-based methodology has been formulated in recent years. Figure 4.4.19 shows a flow chart for the identification of design criteria in the selection of a suitable backfill for local support purposes.

The methodology is as follows:

1. The expected in situ stope width and closure rate should be determined, from which the applicable strain rate can be calculated (See sections 4.4.2 and 10.5.1).
2. *Uncemented backfill* with the lowest achievable porosity should be selected for all high to moderate strain-rate situations, particularly with respect to narrow stopes where self-standing capability is not required and always where cementitious bonds would be damaged. The division between the strain-rate categories is not clear-cut, and it may be necessary in some cases to carry out laboratory tests to clarify the situation regarding damage to cementitious bonds (c.f. note 1 of Figure 4.4.19).
3. In the case of moderate strain rate conditions, it is possible to use either cemented or uncemented backfill depending on whether or not damage occurs to the cementitious bonds. A laboratory test should therefore be carried out, at the in situ strain rate determined, in order to clarify this. Cemented backfill should not be selected if the cementitious bonds would be damaged, except as a means of reducing drainage water in stopes where this is considered to be a problem (c.f. note 2 of Figure 4.4.19).
4. For narrow stopes at shallow mining depths (i.e. low strain rates) when a high early stiffness is required, e.g. in multi-reef mining, and the extra expense is justified, *cemented backfill* should be considered. However, if for example, backfill is required for the purpose of regional and/or 'deadweight' support, an uncemented backfill may be sufficient (c.f. note 3 of Figure 4.4.19).
5. Where stope widths exceed about 2 m in deep or shallow mining situations, cemented backfill should be considered because the strain rates are likely to be low with no damage to the cementitious bonds, and free-standing and low-shrinkage characteristics are required.
6. The mining cycles should be designed to ensure that the backfill is kept within 6 m of the stope face and that appropriate rockfall or rockburst face-area support is installed (depending on prevailing conditions) to ensure the attainment of effective local support at the face.

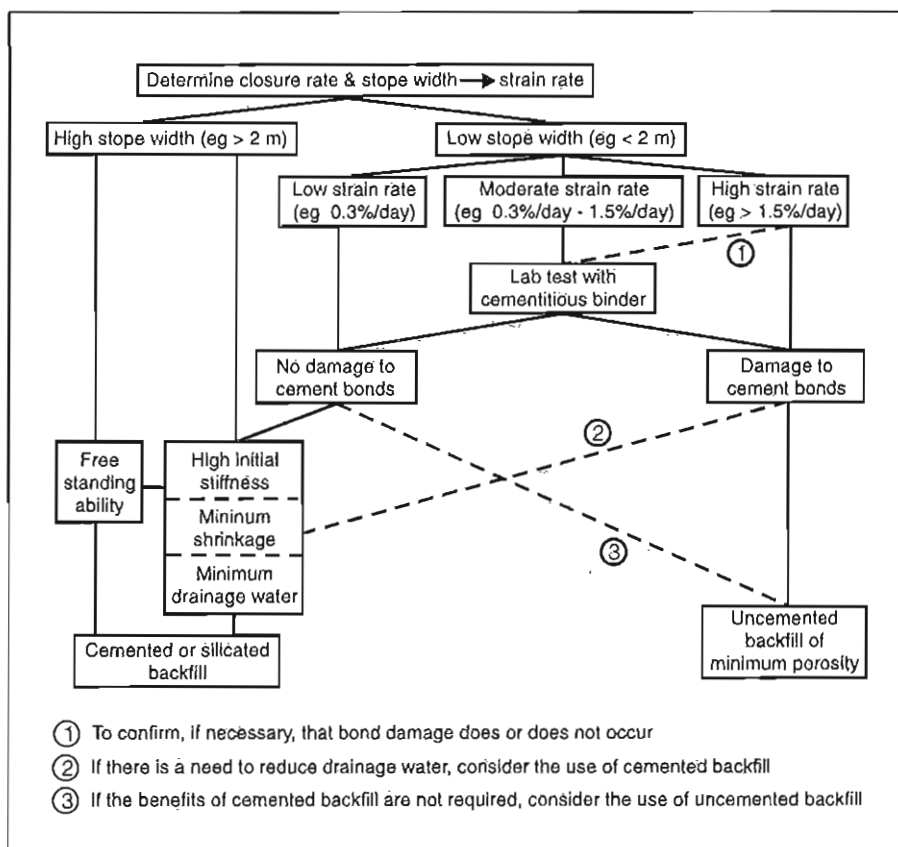


Figure 4.4.19 Design methodology for the use of backfill as local support

A further criterion which should be used for the design of backfill as local support is the percentage of backfill placed in a mined out area. Two surveys were carried out in 1990 and 1994 to investigate the effectiveness of backfill to reduce rockburst and rockfall accidents. Data was collected from mine case studies covering 1088 rockburst and rockfall-related accidents that had occurred during the mining of 3.3 million m² of backfilled and conventional mining. The results of these surveys are given in Figure 4.4.20. What the graph indicates is that the accident rate due to hangingwall instability in backfilled stopes is greatly reduced compared to the accident rate in conventional stopes, provided a significant proportion (60 - 70%) of the mined out area is backfilled. The graph also shows that in panels where only small areas (10 - 30%) had been filled, the rockfall accident rate actually *increased* over that in conventional stopes. Further investigations of these instances showed that the distance between the stope face and the fill was excessive (10 - 12 m), resulting in large spans of poorly supported hangingwall.

Thus, this work indicated that to obtain consistent local support from backfill, at least 60% of the mined out area should be filled and that the fill front should be kept as close to the face as possible, preferably 6 m or less. It is important to note that the percentage backfilling used in this assessment was based on the area of backfill placed in *backfilled* panels over the area mined in those same panels for the same

time period. This should not be confused with an average percentage filled figure for all the mined out areas at a shaft. The correct way of calculating the percentage of backfilling should involve only those stopes where backfill has been placed.

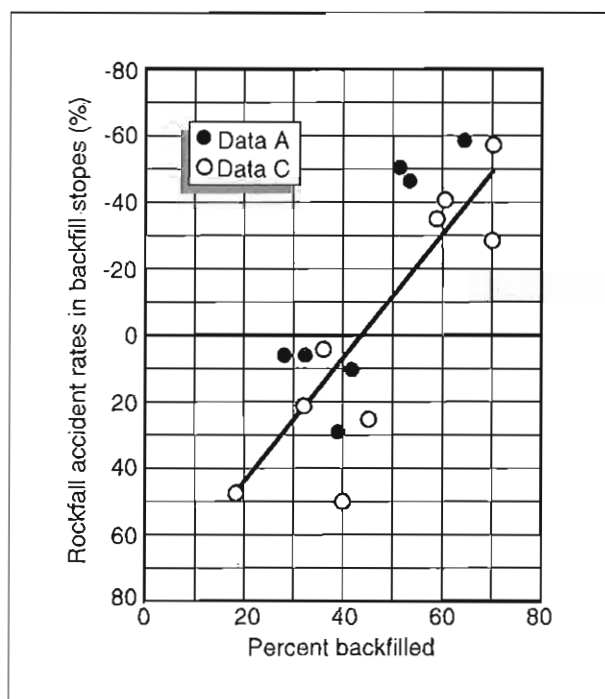


Figure 4.4.20 Effect of percent backfill placed on accident rates, two different time periods A and C. The zero line represents the accident rate in unfilled stopes.

An important aspect of backfilling, which maximises its benefits, is the design and implementation of support for the area between the fill front and the stope face. Apart from meeting the support resistance or energy criteria, the practical and logical integration of this support into the backfilling cycle is vitally important. To achieve this, the strike spacing of the lines of support should preferably be equal to the advance per blast or a multiple of this, using two or three rows of working area support together with the necessary temporary support. In theory, reusable RYHPs together with mine poles are the most cost-effective units to use with backfill. There are, however, some practical difficulties with the use of RYHPs (section 4.5.4), particularly with long load spreaders. More expensive pre-stressable elongates are increasingly being used; these are convenient for confining and securing the backfill bags. They give the added advantage that, wherever poor backfill placement occurs allowing excessive shrinkage to occur, they provide continuous support to the hangingwall beam until back area closure generates normal support resistance in the backfill.

4.4.9 Gully Support

Gully support systems at **shallow depths** should be stiff, and gully spans should be kept to a minimum commensurate with prevailing hangingwall and geological conditions. In competent ground conditions, the use of sticks or cluster packs alone may be adequate. Stiff packs such as sandwich packs, grout packs, elongate support system (EPS) packs, and/or rock tendons may be required in less competent conditions. In common with gullies at greater depth, the gully-stope face area is particularly hazardous and requires sound temporary support. In relatively shallow mines where in-stope pillars are used as part of the support system, low-inclination stress fracturing or induced parting of stratification planes can emanate from the pillar and extend over the gully. Instances of well-bedded sedimentary hangingwall strata, or the pyroxenite hangingwall of the Merensky reef, or chromitite seams immediately above the UG2, can be prone to such problems of hangingwall instability; which arise only at depths greater than a critical value, and where crush pillars are not employed. The fracturing does not usually extend deep into the hangingwall, and can be controlled by roofbolting, designed using the suspension principle.

In **deeper stopes**, where gully geometry is correct, there should be no difference in hangingwall condition between the panel and the gully, and the gully sidewalls should also not present much of a support problem provided no detrimental jointing systems are present. The most effective gully packs are *rectangular* in outline, with the long axis perpendicular to the gully. This shape constrains sidewall dilation and accommodates a degree of sidewall failure without collapse of the pack. Gully packs must be stiff, yet be able to yield sufficiently to accommodate the anticipated closure in the stope without developing forces high enough to cause failure of the gully sidewalls. A compromise between initial stiffness and yield load must thus be reached. A pack constructed from light reinforced concrete bricks has been designed to meet these requirements; having a high initial stiffness and an appropriately low yield load. Timber matpacks may be particularly suitable where large amounts of closure are expected. *Specially-designed* end-grained packs or composite packs can also fulfill the requirements but, like most other gully support systems, can still be prone to premature failure by buckling.

Ideally, gully packs should be installed as close to the face as possible, in conjunction with more active support such as *hydraulic props near the face*. Use of props is particularly desirable in cases where gully pack installation is delayed so as not to interfere with stope cleaning.

Rock reinforcement tendons provide active support for keyblocks in both the hangingwall and sidewalls of gullies - Figure 4.4.21. They should be installed as close to perpendicular as possible to the fracture and bedding planes, thereby increasing the friction between blocks and enhancing the capability of the rock surrounding gullies to be self-supporting. Where deformations are expected to be large, *yielding* tendons should be used. Rock tendons can be used in conjunction with mesh or strapping to increase the area being supported. Serious consideration should be given to reinforcing the sidewalls when the gully is subjected to high stresses; and to reinforcing the hangingwall when the gully width exceeds the normal unsupported spans of the in-stope permanent support system. Where fall-out of the hangingwall between

packs remains a problem, a chequerboard arrangement of packs on the sides of gullies may be tried to overcome the doming of the hangingwall.

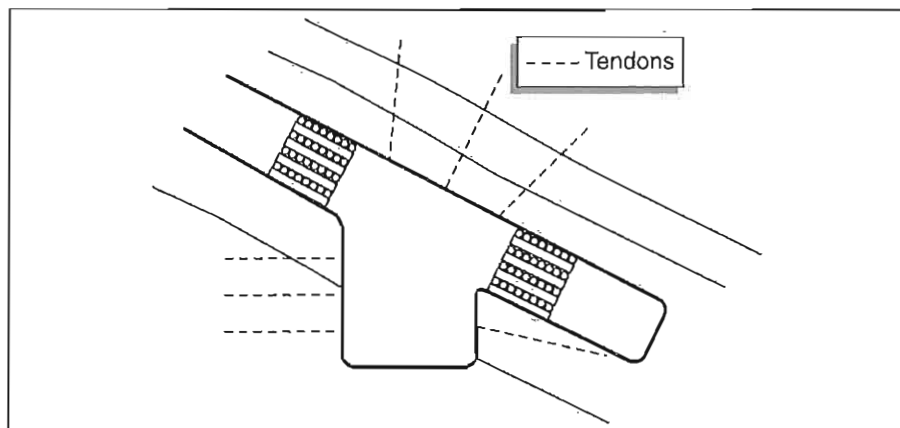


Figure 4.4.21 Gully support in high-stress conditions.

The gully support principles and considerations outlined above apply equally to trackless roadway support, though with greater emphasis where large spans (> 3 m) are involved; in particular, hangingwall reinforcement is normally a requirement.

In **very deep** workings where fracturing is intense, and seismicity levels and stope closure rates are high, considerations such as pack yield load and tendon/fabric reinforcement assume increasing importance, though drilling and grouting operations also become very difficult. Special attention needs to be focused on the critical face:gully area. In particular, gullies sited next to solid abutments, stabilising pillars or substantial leads, require permanent and temporary support with high initial stiffness and areal coverage properties: tendons *and* lacing on the hangingwall and sidewalls, and hydraulic props or elongates near the face area. The influence of adjacent backfill on gully support requirements has been found to be beneficial, especially in rockburst conditions. [In fact, current studies are underway to eliminate gully packs altogether in favour of placing backfill up to the edge of the gully. The backfill here needs to have its stiffness increased by the use of internal weldmesh reinforcement.]

In a recent study of the requirements for the stable layout and support of gullies in very deep mining, the following recommendations were made:

- Lay out the gully on a plan, including local geology. Envisage sidings of at least 6 m against abutments. Create sufficient gully depth for travelling and bunkering, without making the sidewalls unnecessarily high. Ensure correct positioning and alignment by the provision and extension of survey lines, and by the use of painted guide lines underground. Optimise the disposition of blast holes and explosives used to advance the gully.
- Footwall lift the gully as a secondary operation once the gully shoulder support has been installed. This support must have a relatively stiff initial performance (prestressed packs, for example), but must not later develop excessively high reaction forces sufficient to damage the gully sidewalls.
- Support the gully hangingwall, including open areas between gully shoulder supports, with reinforcing tendons and areal surface support. Evaluate the

geotechnical characteristics of the gully sidewalls, and consider the need for similar support here.

- If the area is likely to be seismically active, select support units which will perform satisfactorily under dynamic loading conditions.

Problems of bulk movement of the sidewalls of dip gullies or hangingwall buckling failures in more ductile rock masses can be inhibited by cutting slots in the footwall or hangingwall adjacent and parallel to the gully - c.f. Chapter 3, Figure 3.4.19.

4.4.10 Application Of Rapid Yield Hydraulic Props and Prestressable Elongates

A rapid yield hydraulic prop (RYHP) system comprises a pneumatic pump delivering fluid at 15-20 MPa via high pressure hoses to a setting pistol which is used to pressurize the cylinder/ram steel props. Early props used a 5% emulsion as the hydraulic fluid, but current props operate on water alone and can thus be easily integrated with hydropower systems. The props can be remotely set and released.

RYHPs were a South African innovation to cater specifically for the high velocity of closure experienced in some rockbursts. The critical component was thus a control valve which would open at a predetermined pressure, allow rapid expulsion of hydraulic fluid at closure rates of > 1 m/s, and then close off whilst maintaining almost the original pressure as the rapid closure ceased. This development, which occurred in the late 1960s, gave rise to a group of competitive manufacturers of these sophisticated props, and there have always been at least three suppliers in the market.

Important accessories supplied with the props are extension pieces which, together with a range of prop lengths, allow RYHPs to be used successfully in stoping widths of up to about 1.8 m. An essential component of a prop system is a headboard (load spreader). These are supplied in lengths of up to 800 mm, and not only significantly increase the effective areal coverage of the support but also provide enhanced stability to the prop and blast resistance, as well as allowing the prop to be effective in areas of friable ground. In addition, the props can be used to hang or support blast barricades relatively close to the stope face, thus improving the muck cleaning operation.

As part of the development in the early 1970s, field trials and research were carried out on how best to implement and organize hydraulic prop systems on a mine. Comprehensive guidelines were issued covering such issues as organizational structures, maintenance requirements, repair workshops and testing, other infrastructural requirements, control methods and training of support crews. With these guidelines in place and good operator acceptance of the new system, implementation spread rapidly through the industry and those mines adhering to the guidelines achieved utilization factors of over 80%, and an operational prop life of about 7 years. The population of props on the mines peaked at about 250 000, implying that at that stage approximately 100 km of stope face was supported on props.

In 1988 the original prop specifications were reviewed in the light of a better understanding of rockburst support requirements, and to introduce some desirable practical improvements. These included the ability to run the system on pure water, to

reduce the mass of a prop from 50 kg to less than 37 kg (a figure based on ergonomic research), and to develop improved load-spreaders and extension pieces. In addition, the props were to be blast-resistant in so far that they could be installed less than 2 m from a stope face without suffering significant damage or being blasted out, and should be capable of resisting closure rates of at least 3m/s.

All these requirements were achieved and verified through extensive field trials, during which a highly effective face area support system design was developed and implemented. This involved the integration of water jet assisted face scraper cleaning, diagonal/overlapping blast barricades and the use of props with improved load-spreaders and extension pieces where necessary. The outcome of this trial was *significantly* improved stope width control, more accurate shot-hole drilling and better face advance - c.f. section 4.5.6.

Figure 4.4.22 compares the performance specifications of the original and current hydraulic props. The main differences include the lower slow yield force, and the more stringent rapid yield force. This performance had to be achieved at a compression rate of at least 3 m/s, compared to the original 1 m/s.

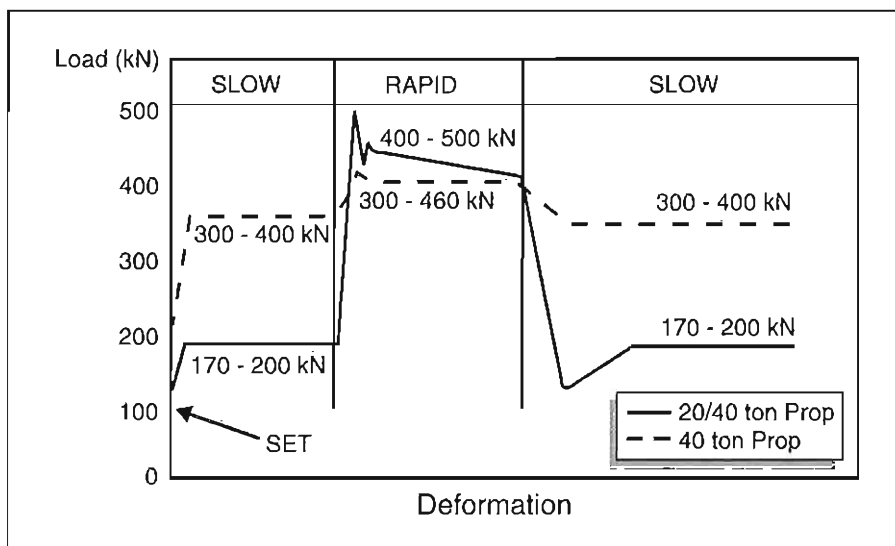


Figure 4.4.22 Comparison of performance specifications of original (40 ton) and current (20/40 ton) RYHPs.

The main purpose of the lower slow yield force was to lessen the stress on accessory components during the great majority of the props' installed life; to easily provide a support resistance of 50 kN/m² sufficient to prevent the majority of rockfalls; and to minimize damage to the hangingwall by not applying too high a force. These props became available in 1992, since when over 200 000 have been supplied to the mines. They can be set less than 2 m from the stope face, and are able to provide safe and cost-effective face area support for stopes, especially in those using backfill.

Unfortunately, in order to realise these advantages of the use of RYHPs on a mine, it is essential that a number of special control structures be in place and operate efficiently (failing which large numbers of props can very rapidly go 'missing', or fail to deliver adequate levels of safety or mining performance):

- Prop maintenance facilities (underground or on surface) to provide periodic maintenance according to manufacturer specifications.
- An effective training programme for operators.
- Regular prop installation monitoring, on a 'random' but controlled basis, to ensure adherence to in-stope usage standards, as well as to confirm the physical existence and location of sets of props.
- Occasional laboratory rapid-yield testing of batches of older props, to ensure they continue to perform according to specification.
- A (computer-based) **prop control system**, in which the physical location and maintenance history of each individual prop is recorded and updated, in order to coordinate, expedite and monitor the above activities.

These facilities are not only fairly costly and skill-intensive, they are foreign to the normal activities of production mining. Recent years on the mines have seen a consequent growing disenchantment in the in-house use of RYHPs. Rather - and not unlike similar trends in shaft-sinking or backfill placement - mine management are increasingly seeing RYHP deployment and control as the province of specialized outside *contractors*. These organizations have been able to achieve efficiencies at least equal to the best in the past (a successful innovation, for example, has been the use of transponders to automate the identification of specific prop units underground), and it is claimed the stopes in which they operate have an enviable safety record.

At the same time that these trends were evolving, a new generation of yielding elongates became available for in-house use in deep mines. These prestressable elongates (PSEs) provide a viable alternative to RYHPs; their fundamental advantage being that they do not have to be removed for reinstallation, nor do they require maintenance or geographic control.

There are, nevertheless, two technological disadvantages of most current PSEs with respect to RYHPs: these have to do with areal coverage and consistency of performance. Firstly, no loadspreader for PSEs approaches the strength and dimensions of those available for hydraulic props, and the potential to develop significantly improved areal coverage by solidly linking props appears limited with PSEs when considering rockburst conditions. Secondly, as the majority of PSEs are based on timber props, their consistency of performance cannot approach that of hydraulic steel props - Figure 4.4.23. This issue is currently being addressed by a substantial testing programme and statistical analysis of results, which will provide a performance characteristic for each type of PSE guaranteed to some probability level of achievement. This 'nominal' load/deformation characteristic can then be used by the support design engineer with some confidence.

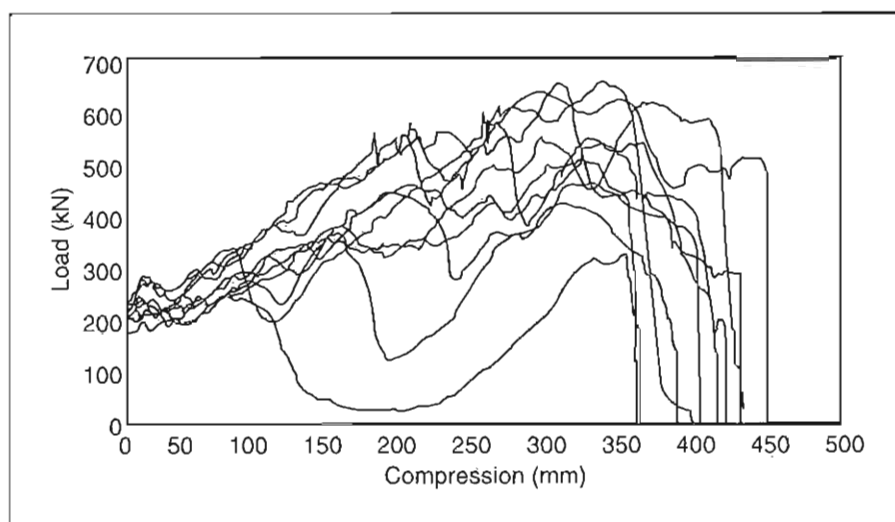


Figure 4.4.23 Variable performance of a particular type of timber-based elongate. [Steel clad PSE's exhibit lower variability].

A final (economic) aspect has to be considered when deciding on whether to use RYHPs or PSEs. Properly managed, RYHPs deliver the best face area support available in deep rockburst-prone stoping. Where *backfill* is being placed (or else, with a few extra rows of RYHPs together with the cheapest form of permanent support elongates), more than adequate face and back area support can be realized. Since RYHPs can be re-installed more than 200 times, the cost indications of such a support system would appear to be extremely favourable.

It is true, nevertheless, that running an efficient hydraulic prop system on a mine requires efficient and specialized maintenance, training, monitoring and control structures to be in place. If there are deficiencies in any of these aspects, the effectiveness of the system will decline. Thus, the option of using **contractors** to supply a cost-efficient service in the use of RYHPs should be carefully considered by any deep mine confronting a significant incidence of rockbursting.

4.5 EXAMPLES OF PANEL SUPPORT SYSTEMS

In this section a number of support systems that have been used on gold and platinum mines are depicted in the figures, and the advantages, potential deficiencies and variations are discussed. The final two ('cave mining' and 'RYHP diagonal blast barricades') are examples of innovative systems which promote close-in face area support; maximizing safety along with excellent productivity performances.

4.5.1 Reef Pillars with Simple Timber Prop Panel Support

These systems are used in **shallow** mines, with no stress fracturing ahead of the stope

face and tensile stresses in the immediate hangingwall. Potential failure mechanisms include major backbreaks, large panel collapses (beam or plate failure), and the fall out of individual unstable blocks or keyblocks. All the above mechanisms are usually associated with geological structures of unfavourable geometry, but in some instances mining spans that are inappropriately large for the conditions can also contribute to instability.

Backbreaks and, to an extent, large collapses are prevented by the design of appropriate in-stope pillar systems. At depths shallower than about 500 m, depending on geotechnical conditions, the most effective design ensures that the pillars are not subjected to stresses that exceed their elastic range. Where barrier pillars are not used, the in-stope pillars, together with unmined geological loss areas such as potholes, fault losses and dykes, must bear the full cover load. At greater depths crush pillars can be used to good effect - Figure 4.5.1. The dimensions of these

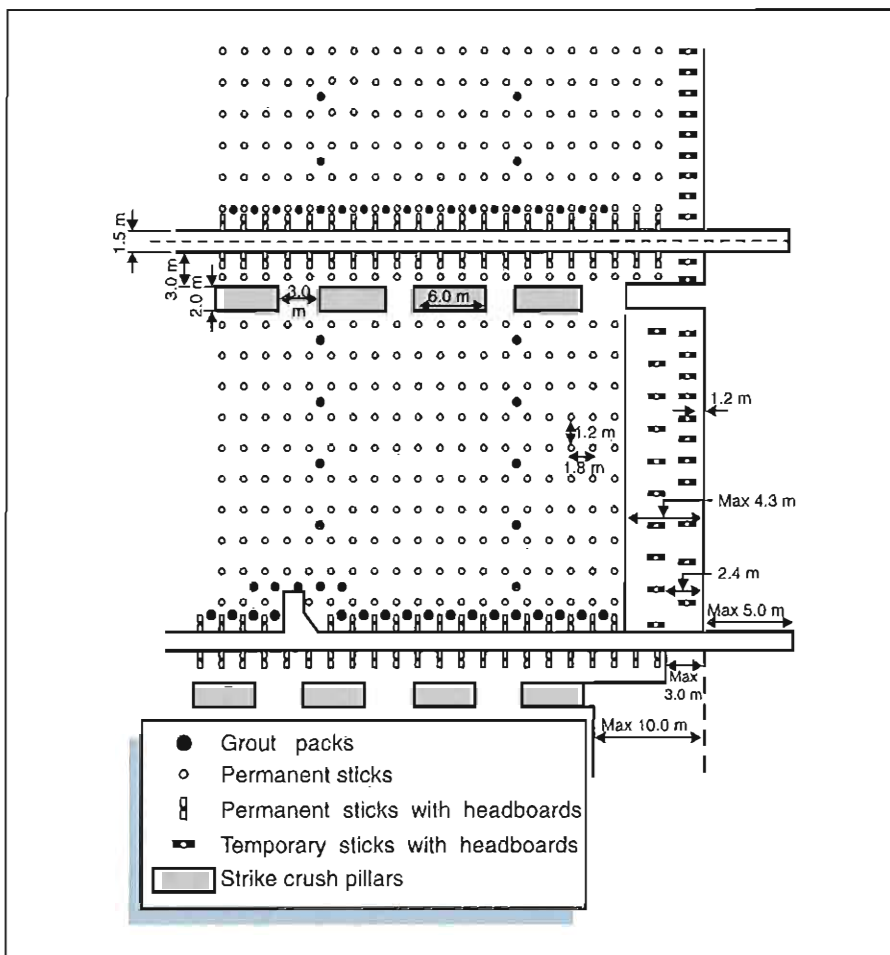


Figure 4.5.1 Typical shallow mining support layout, utilizing strike 'crush' pillars.

pillars should be such that when cut at the stope face, they should be thoroughly fractured and at their residual strength. The hazard associated with not complying with these design criteria is possible violent failure of the pillars in the stope. The span between pillars or panel length is often dictated by mining requirements and past experience. This bears the risk of unexpected changes in geotechnical conditions, particularly the intersection of a persistent strike-oriented joint or fault, which, without the inclusion of additional pillars, could lead to a panel collapse.

Adequate in-panel local support is commonly provided by timber props. A disadvantage sometimes encountered is that, as the support cannot be installed close to the face, falls of ground in the face area can occur in unfavourable conditions, such as 'domes' in the hangingwall or near potholes. This problem can be overcome by the installation of roof bolts close to the face or by prestressing the props or elongates such that they can be installed 2 m or less from the face. The problem of premature failure of props caused by footwall heave, resulting from pillar punching, can be overcome by installing yielding elongates.

Large scale toppling or ride-out failure of props leading to collapses is difficult to anticipate. Where the likelihood of such a failure is suspected, the collapse can be prevented by substituting rows of props by packs or grout packs on a regular basis. The use of cluster sticks is also a partial solution by providing greater support resistance and enhanced yieldability. Use of these types of support is recommended for stoping widths in the range 1.8 - 3 m, as the risk of prop buckling is increased at high stoping widths. For stoping widths greater than about 3 m, roof bolting is usually necessary with reduced inter-pillar spans, or backfill can be used.

4.5.2 Timber-Based Elongate Support Systems

The large family of elongate support units, introduced in the gold and platinum mines over the years, was developed to accommodate stope closure rates in excess of those tolerable by normal timber props. Yield capabilities of 200 mm to over 400 mm are typical. These systems are used predominantly in medium to deep stopes where the rockburst potential is moderate - Figure 4.5.2. However, they also find application in shallow stopes where the rates of closure are abnormally high for these depths. Often, steel clad elongates ('pipe-stick' type) can be used successfully in stopes where the risk of rockbursts is relatively high. Currently, about a third of all stopes are supported by these cost-effective systems.

The main function of elongate support systems is to prevent rockfalls in stopes where the hangingwall is highly fractured and the closure rates are moderate to high. Nevertheless, they are also required to contain rockburst damage. To fulfil those functions the systems must provide:

(i) Adequate support resistance and energy absorption - these are readily achieved at spacings of approximately one prop per 3 m². These spacings must however be carefully considered by taking into account the often high variability in performance between different units, and a reasonable factor of safety should be employed. [A current research programme is underway to establish standard load/deformation curves and appropriate safety factors for each of the many variants of elongate on the market]. Energy absorption capability is enhanced in timber-based units, because the

resisting force increases at high rates of closure (rockburst conditions). The opposite is, unfortunately, the case for types where frictional resistance is used to control the yield force.

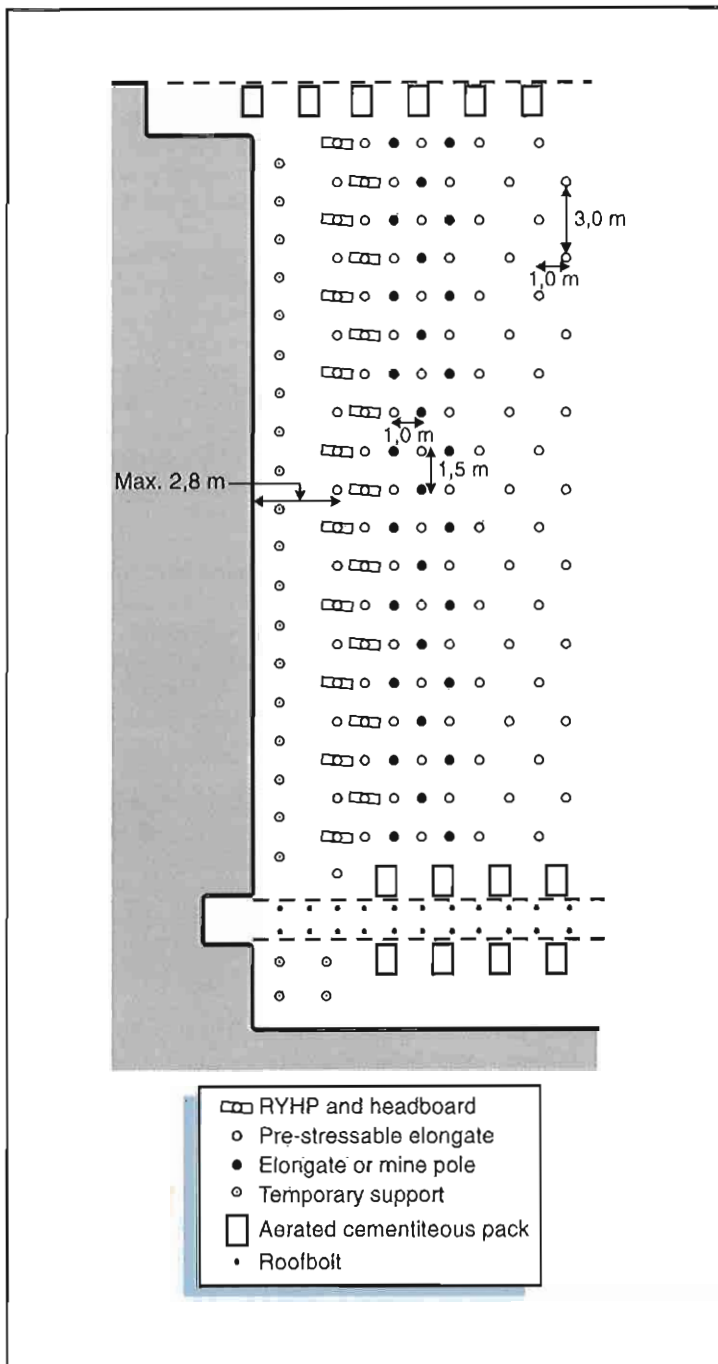


Figure 4.5.2 Stope with a primarily elongate-type support system

(ii) Adequate areal coverage. A perceived weakness in prop-based systems is the relatively low direct areal coverage provided by the system. [However, as a better understanding of the interaction of elongates with the hangingwall strata and each other is gained and the concept of 'area of influence' is quantified, this reservation may be relaxed.] Thus, an empirical derivation of stable spans between units needs to be determined for different geotechnical conditions. The common practice of having the strike spacing greater than the dip, to accommodate scraper cleaning in the back areas, is strongly discouraged on the basis of the dominant discontinuity direction being (usually) stress fractures parallel to the face. Attempts to improve the areal coverage provided by elongates by means of headboards have met with limited success. The mode of yield failure of most elongates is not conducive to the attachment of effective headboards, particularly for dynamic loading. Nevertheless, any effort to improve areal coverage is beneficial and should be pursued.

A recent significant innovation in the field of elongate support has been the development of pre-stressing devices and elongate designs which allow the support units to be installed within 2 m of the stope face without the risk of being blasted out. This addresses the very important issue of having good support in the face area where the majority of accidents occur, especially the problem of inadequate support in the face area during cleaning operations. An added benefit is that these units are installed as face area support, and then act as both working area and back area support. Thus, only one type of support (apart from temporary face area support) is required, simplifying infrastructure and training requirements and eliminating the complications of integrating a second type of permanent support.

The disadvantages of pre-stressable units are their limited area coverage, particularly under rockburst conditions, when compared to RYHPs with adequate load spreaders; the variability in performance of most of the types available; and the relatively high cost of the units particularly when used with backfill. It is nevertheless considered that the development of this type of support unit was an important advance in support engineering, giving practitioners the opportunity of providing improved support designs in many geotechnical conditions.

An interesting aspect of elongate support systems is the observation that in areas of high closure rates (i.e. 'deep' mining conditions), the majority of units *fail completely* some 30 m behind the face, without any apparent detrimental affect on stability. This poses the question of how necessary is the presence of high-quality high-cost back area support in these conditions?

4.5.3 Pack Support Systems

A large variety of pack types are available, with widely diverse load-deformation characteristics. The options range from relatively soft and weak skeleton packs, through mat and solid timber packs, combinations of concrete blocks and timber (composite and sandwich packs), grout packs with the grout gravitated from a surface preparation plant, combination of elongates and a framework of timber slabs (EPS packs), to the more recent innovations of combinations of end-grained timber blocks with parallel-grained slabs and lightweight foamed concrete, reinforced with either annealed steel mesh or wood fibres. These latter two types are supplied in modular sections, which interlock and can be engineered to provide a wide range of combinations of initial stiffness, yield force and strength. Thus, the challenge to the design engineer is to define the support requirements, and then to choose the appropriate pack type to match the requirements in the most cost-effective support system.

Packs are used as panel support in many geotechnical conditions and almost univer-

sally, except in some shallow platinum mine stopes, as gully support. They comprise about 60% of support used in gold mine stopes, but play a relatively minor role in platinum mines where stiff low-yieldability support is generally required. The characteristics of packs, which account for their widespread usage are: high strength, at least 50% yieldability in most cases, structural integrity when subjected to eccentric loading, and relatively large direct areal coverage compared to prop types.

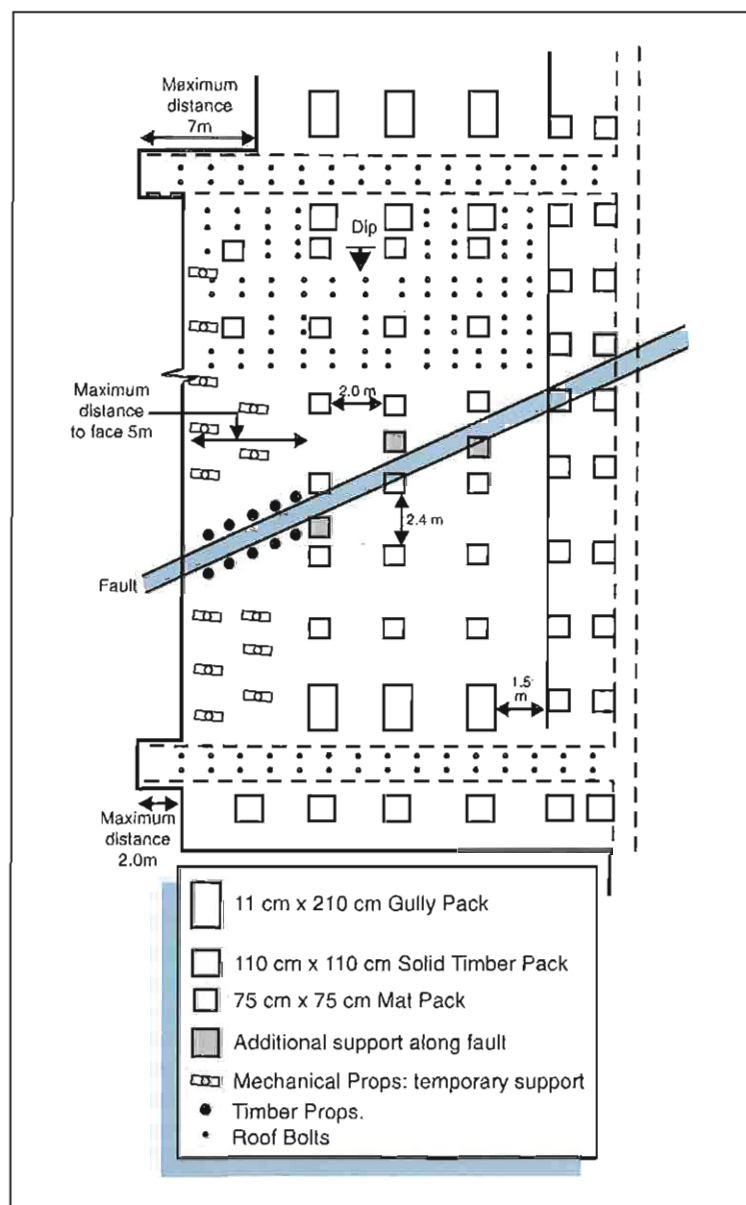


Figure 4.5.3 Typical pack-based support system (rockfall control only). Integrated with rockbolts in upper half of diagram.

The benefit of the areal coverage is somewhat offset by the relatively large pack skin-to-skin spacings often employed, leaving unsupported spans of 2 m to 4 m. The other disadvantage of the wide spacing of packs is that they cannot at all times be kept close to the stope face. Where two or more rows of hydraulic props are used ahead of the packs in the face area to overcome this problem, the disadvantage is converted into an advantage. Roof bolting the hangingwall close to the stope face is another effective means of overcoming this problem in certain geotechnical conditions, and particularly in higher stoping widths. In blocky or weak hangingwall conditions, closely-spaced stiff pre-stressed packs are the preferred support. Packs are also probably the most effective way of stabilizing weak geological structures cutting through a stope.

Figure 4.5.3 shows an example of a pack support design for average hangingwall conditions where there is *little expectation of rockbursts*. The stage in the mining cycle that is depicted is when the new line of support is due to be installed. Two rows of labour-intensive temporary support are required in the face area. In the prior stage, particularly during cleaning and barring operations, the face area is subjected to an increased rockfall hazard and strict adherence to standards is required. The use of hydraulic props in advance of the packs would greatly reduce this hazard, as would the installation of roofbolts as depicted in the upper half of the diagram. The additional support required adjacent to faults is also shown - note the substitution of timber props for mechanical props in the face area to eliminate the hazard caused by removing the mechanical props prior to the blast in this potentially unstable situation.

4.5.4 Backfill Systems

The selection of backfill type and issues concerning the placement of backfill were dealt with in sections 3.3.5 and 4.4.8. In this section, discussion is centred on the working area support used in conjunction with backfill. The requirements for this support are no different to those needed with any other permanent support types. The key issue for a successful overall system is the precise integration of the working area support with the backfill placement cycle requirements. This involves both the spacing, and the timing of installation and moving, of rows of support.

Important parameters, which have to be decided first are:

- (i) The maximum and minimum distances that the backfill front will be kept from the face. An absolute maximum, after the blast, of 7 m is indicated if three rows of support are to be used and proportionately less for fewer rows.
- (ii) The average face advance per blast achieved in the area under consideration.
- (iii) Whether backfill is to be placed after every blast, or after every second blast. The accumulated experience gained since the reintroduction of backfill into the mines using modern technology is that, for low to moderately high stoping widths, the placement of backfill in geotextile bags is the preferred option. [The use of paddocks is appropriate for high stoping widths, and is also indicated for situations where large areas need to be backfilled in one operation (e.g. the filling of an up-dip panel after holing into the panel above or achieving the stoping distance)]. This third parameter, together with the stoping width, will determine the size and width of the filled bag. The bag size selected should take into account the maximum expected combination of stoping width and face advance per blast to ensure complete filling to the hangingwall at all times.

The maximum distance of backfill to face will define the number of rows of support required. This decision can, however, be taken in reverse order. The average advance per blast will define the strike spacing of rows of support, and hence the width of the backfill bag.

It is clear that some iterative modification of the above parameters will be required to arrive at the optimum design. It is important that some flexibility be allowed in the strike spacing of the support. This should equal, or be a multiple of, the day to day advance achieved, otherwise the first row of support will be forced either too close or too far from the stope face. The key to a successful and safe backfill system is this close integration between support spacing, face advance and consistent backfill placement.

The most cost-effective working area support to integrate with backfill is a hydraulic prop system, as the units are reused. However, there are a number of practical difficulties. The valve cannot be ideally oriented away from the blast, as it would be buried in the backfill when in the back row; current headboards tend to become trapped in the backfill; and there is the potential for increased wear rates of props from backfill entering valve housings and seals if flushing procedures are not strictly adhered to. These problems can be alleviated if the timber props, used to support the backfill bags, are placed not in line with the hydraulic support but some 200 mm to 360 mm behind them - see Figure 4.5.4.

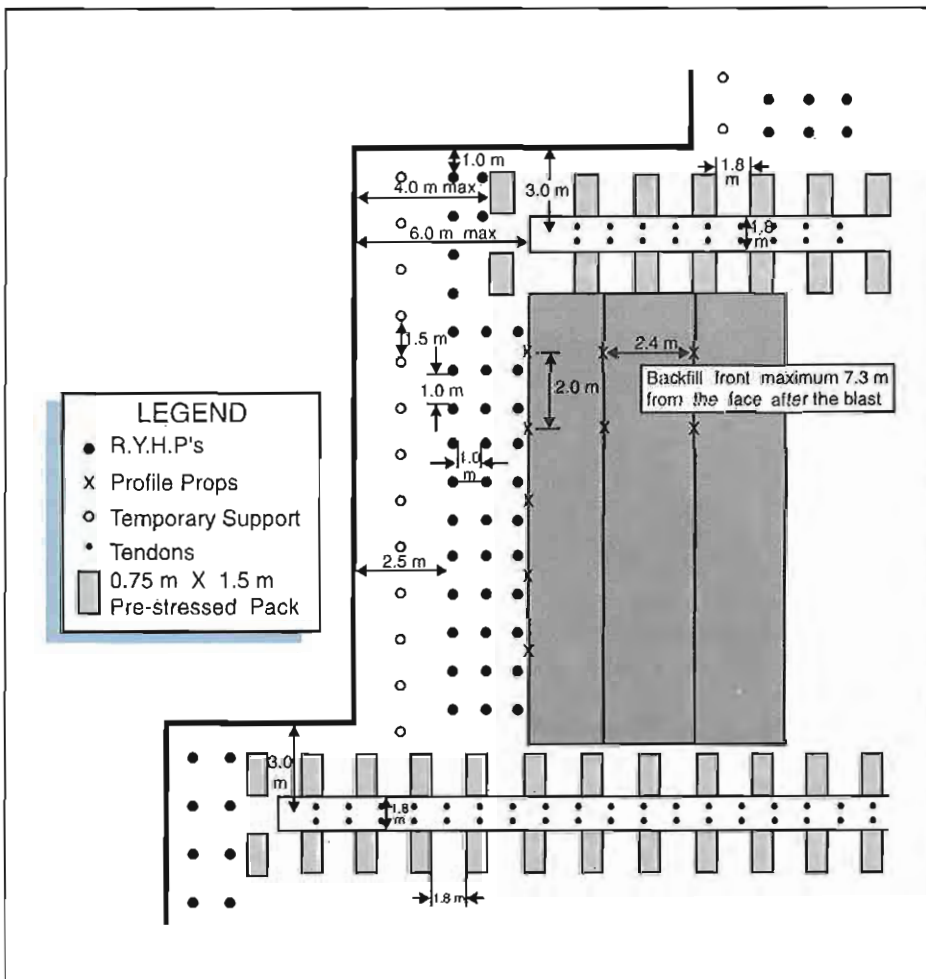


Figure 4.5.4 A backfill support system employing three rows of hydraulic props.

Apart from the above considerations, the design of the support follows the same guidelines as discussed elsewhere in this chapter.

[Modifications to *gully support* in backfilled stopes is presently under investigation. Currently, standard designs of packs on gully shoulders and roofbolts in the gully hangingwall are employed. It is likely that significant stability and logistic benefits would be gained by eliminating the gully packs, and bringing the backfill to the edge of the gully. At present, some field trials of such a system are underway to identify problems and to assess benefits. In these experiments, mesh reinforcing of the backfill adjacent to the gullies to improve stiffness and reduce bulging into the gully, is being evaluated.]

4.5.5 Cave Mining Support Systems

Successful cave mining is restricted to well bedded strata with low-friction bedding planes - section 3.5.5. The system comprises a well-supported working area with no back area support except along gullies. It is important that at least the first few hangingwall strata cave on a regular basis behind the support. The design of support for cave mining has to achieve a balance between two requirements - sufficient support resistance to prevent loosening of strata to a height determined by local geological conditions, yet a resistance not so high as to prevent sliding of the strata. This horizontal shearing has the effect of relaxing the horizontal stress which otherwise could cause buckling and other deleterious effects to this type of hangingwall. As even a distribution as possible of the support resistance across the supported area needs to be achieved, implying that the first row of support needs to be placed as close to the face as possible.

Cave mining has been carried out using both timber props and hydraulic props derated to yield at 300 kN. With the former, four rows of props are used, the fifth being blasted out if not failed, to keep the cave front about 7 m behind the face - Figure 4.5.5. The hydraulic prop layout employs only three rows, but at a wider spacing.

It is important to the success of cave mining that the cave occurs regularly as, even under these relatively thin-beam conditions, the hangingwall can be self-supporting and quite often it is necessary to blast the hangingwall to induce a cave. This initial collapse is only required to extend a metre or so into the hangingwall. The cave occasionally can over-run part of the back line of support but is invariably stopped by the next line. There have been virtually no instances of the cave over-running the gully support, which comprises two rows of stiff packs arranged in chequerboard fashion together with roofbolts in the hangingwall over the gully. The installation of gully packs must be kept ahead of the cave line. Indications that conditions are deteriorating are given by difficulty in drilling shot holes, an increase in closure rate and change in fracture orientation. Often this is caused by a cave that is hung up, and conditions can be relieved by inducing the cave and ensuring a regular face advance.

The cave mining system not only provides relatively safe mining, but the discipline required for its implementation and the indicators of when remedial action is required are conducive for high efficiency and productivity (Chapter 3.5.5). Cave mining is a good example of how a thorough understanding of the rock mass behaviour around a stope can lead to a successful design of not only the support, but also the whole mining system.

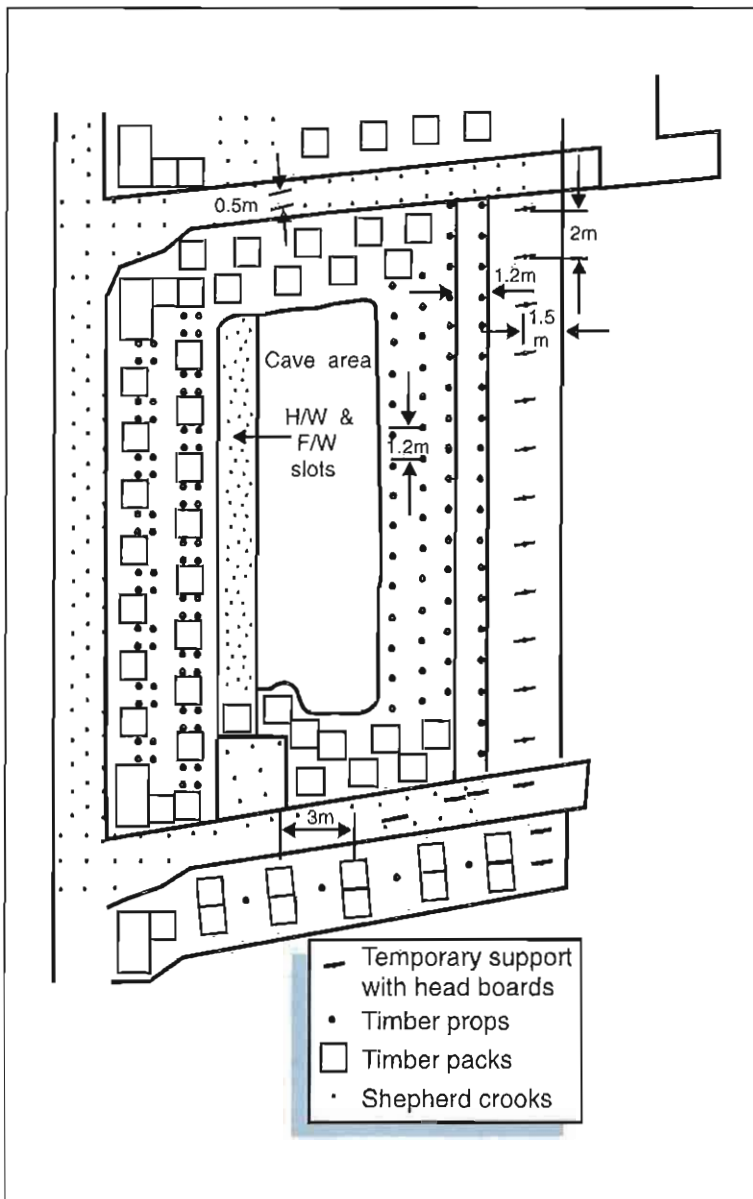


Figure 4.5.5 A support system for cave mining

4.5.6 Rapid Yield Hydraulic Prop System With Diagonal Blast Barricades

This innovative and quite sophisticated integrated system was designed to cater for all rock conditions, including dynamic loading. The objectives of the design were to provide a safer working environment, improved stope width control (reduced dilution), and increased productivity. To achieve these objectives, the principal requirements were to enable the first row of support to be installed no more than 1.5 m from

the face, and to provide nearly continuous areal coverage in the direction normal to the stope face, thus maximising support of the face-parallel fracture set.

The system comprises four rows of 200/400 kN rapid-yield blast-on props equipped with 800 mm load-spreaders (headboards). The props in adjacent rows are offset such that they delineate an angle of 70° with the face (the design angle for drilling shot holes in the face). This has the consequence of improved drilling accuracy with resultant greater advances per blast. The prop rows are spaced at a distance on strike equivalent to the advance achieved per blast, and the props are spaced at 1.5 m in the rows - Figure 4.5.6. The load-spreaders are angled at 20° above strike, to face into the throw of the blast in order to reduce the area subjected to fly rock. This layout ensures that drilling and other face activity takes place under the protection of active support with good areal coverage.

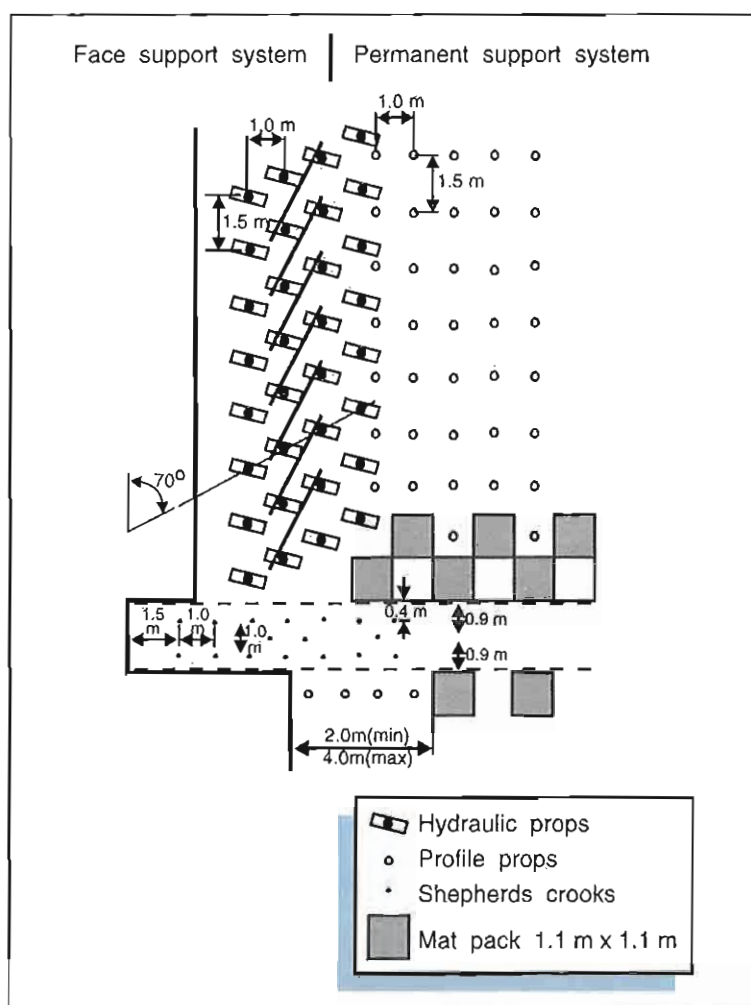


Figure 4.5.6 Layout of rapid-yield hydraulic prop system with diagonal blast barricades

Another essential component of the system is overlapping **diagonal blast barricades**, which are attached to props in adjacent lines in the second and third row back from the face. This innovation makes the system viable, as barricades parallel to the face on the first row of props would result in choking of the face after the blast, and, if hung from the second row of props, would make it difficult to clean the broken rock from between the props. The diagonal barricades allow cleaning to take place by means of **water jetting** between the gaps in the barricade, flushing the rock into the scraper path. In addition, the water jetting efficiently performs the task of sweeping simultaneously with the cleaning. A fines barricade is kept on the fourth row of props.

Where this system was used, the casualty rate dropped by over 35%, the stoping width was reduced by 10-17%, and the face advance rates ranked in the top 10% in the area.

The motivation for using such a system is clear. It can be implemented with any type of back area support strategy from backfill to caving. This system also demonstrated that successful implementation requires effective training and teamwork of the mining crew, and good supervision of standard procedures which evolve through experience gained by supervisors and operators alike.

4.6 STOPE SUPPORT IN SPECIAL MINING SITUATIONS

Definitions and layout procedures related to special areas (remnants, geological hazards, etc) were given in Chapter 3.5, and should be referred to.

4.6.1 Remnant Support

Remnants are highly stressed blocks of ground, and the density and quality of stope support must be upgraded from that normally used, in order to cater for increased hangingwall fracturing, rates of closure and the likelihood of rockbursts. This may be achieved in one or more of the following ways:

- (i) reducing the strike and/or dip spacing between support units (both stope and gully support)
- (ii) reducing the support-to-face distance to an absolute minimum
- (iii) increasing the density of temporary face support units
- (iv) using headboards, and/or umbrellas to achieve greater areal support coverage
- (v) pre-stressing permanent support units
- (vi) installing stiffer support units with higher support resistance
- (vii) introducing rockburst-resistant support types such as RYHPs, PSEs and/or barrier props
- (viii) employing backfill to improve hangingwall strata conditions and reduce the effects of rockbursts.

Remnants should be identified on stope sheets and mine plans to ensure they are given adequate attention during mine planning and operational meetings.

4.6.2 Support of Stopes near Geological Structures

Faults and dykes constitute geological weaknesses in which the potential for falls of ground or rockbursting is enhanced, and special support precautions in excavations situated in or near them are often required.

The principles governing stope support near geological structures are relatively simple:

- (i) additional support is required adjacent to and within geological discontinuities and dislocations; in particular weak zones should be recognised and adequately supported
- (ii) both sides of such structures should be supported, but particular attention should be given to supporting the weaker side of hangingwall discontinuities; in general, the possibility of the formation of unstable keyblocks should be recognised and countered
- (iii) particularly high standards of areal coverage of permanent support should be used in high risk areas such as travellingways and gullies, and of temporary support in the working face area wherever geological structures are encountered.

Geological structures should be identified on stope sheets and mine plans to ensure that adequate attention is paid to them during planning and operational meetings.

4.6.3 Stopped Panel Support

During normal stoping activities, panels are advanced on a regular basis and stope closure takes place at a rate determined by the stope depth, mining geometry and rock properties. This closure is accommodated in the stope face area by support units which have been designed to meet these normal conditions.

However, stope closure is highly time-dependent, particularly in deep stopes where movements of a stopped panel can persist for some weeks before finally stabilizing (Chapter 1, Figure 1.3.10). Hangingwall conditions in stopped panels can deteriorate as a result of the prolonged relaxation of discontinuities in the surrounding rock mass (bedding, fractures, jointing, etc.). This unravelling is generally not marked in shallower workings, but can be a serious problem if faces are not advanced regularly at depth. In addition to the general deterioration in ground conditions, support units can suffer damage. This is particularly serious in the case of rapid-yield hydraulic props, which freeze solid when their travel is exhausted, punching into and further damaging the hangingwall and footwall. To avoid this damage and the dangers associated with the retrieval of props under these conditions, it is good practice to replace them with stiff support (for example, solid packs, or a pack/elongate combination) if a panel is to be stopped for any reason.

Thus, if a prolonged stoppage is anticipated, additional stiff support should be installed close to the face to contain deterioration and facilitate access for re-start operations. This applies to prop-supported and pack-supported panels alike. These support measures will, however, not reduce or stop the time-dependent closure, and may only marginally limit the associated unravelling of the hangingwall. The formation of an intensified fracture zone ahead of stopped panels will also continue, and due care must be taken when such panels are subsequently re-started (see following).

4.6.4 Support while Re-Opening Panels

Once a panel has stopped for any period of time, a zone of intensified fracturing is formed ahead of the face. The extent and intensity of this zone increases with increasing depth, and also with the period for which the faces were allowed to stand.

During re-start operations, due caution should be exercised and additional support installed while this zone (which can extend up to 10 m ahead of stopped faces at depth) is being negotiated. Support-to-face and inter-support spacings should be reduced, and it may be necessary to increase support stiffness. In extreme circumstances, short rounds should be drilled when advancing the stope faces through these zones so as to minimise additional blasting damage. In general, deterioration and resulting hazards increase with increased standing time. It follows that, if panels are stopped for any reason, they should be re-started as soon as possible, particularly at depth.

Re-establishment of *collapsed panels* may be achieved within the confines of the existing excavation (open-raising) or by re-raising in solid ground, with or without re-development. Support requirements for open-raising in collapsed areas tend to be ad hoc and dictated by the prevailing circumstances. As a general rule, intensified support density and stiffness of support units is required. Pack height:width ratios should not exceed 2:1, and pre-stressing of pack support should be considered. Where the remaining hangingwall is blocky, some type of umbrella support is advantageous. Particular attention should be paid to temporary support while clearing operations are in progress.

If re-raising is carried out, it is generally good practice to carry wide-ends, particularly in intermediate and deep mines. Support-to-face and inter-support distances should be kept to a minimum, and it is generally good practice to use stiff support units in these circumstances. Support of strike and dip gullies within the wide-ends should similarly be at least as intense as that required for conventional gullies in the vicinity.

4.7 STOPE SUPPORT FOR SPECIAL OREBODY GEOMETRIES

Unconventional mining methods which arise as a result of the nature of the orebody itself include wide-reef, multi-reef, and steeply dipping operations.

4.7.1 Wide-Reef Stope Support

Wide-reef mining is generally accepted as that in which the stoped reef width exceeds 2.5 m. Mining heights can be very large, sometimes requiring massive mining techniques.

Three principal types of mining methods are used to exploit wide-reef orebodies: conventional mining, pillar mining, and massive mining. Support systems employed vary depending upon the mining method used.

In *conventional mining*, normal timber-based support units become increasingly inefficient with increasing height, particularly in terms of stiffness which decreases inversely with the height of the support unit. There is also a greater potential for these units to buckle with increasing height; packs for example should not be constructed with a height:width ratio exceeding approximately 2:1. For mining heights in excess of 2.5 m and up to 3.5 m, use of packs larger than 1.1 m x 1.1 m, or of clus-

ter elongates, should be considered. Large grout packs have also been successfully used at these mining heights, though particular attention has to be paid to filling the potential gaps between the top of the grout and the inclined hangingwall surface. In shallow and intermediate depth wide-reef conditions, prestressing of support elements becomes desirable to overcome the effects of low stope closure rates and decreased support unit stiffnesses.

Wide-reef *partial extraction (pillar mining)* is not generally practiced deeper than 1500 m, due to unacceptably low extraction ratios with increasing depth. A primary consideration for this type of mining is the support of the rooms or bords between the stope pillars, usually by some form of roof-bolting system. The support density and length of the roof-bolts are important factors in the design; these are mainly governed by the mechanical properties of the hangingwall rock, the room or bord width, and the existence of geological discontinuities such as joints, faults or bedding planes in the hangingwall. The density of roof-bolts should increase in relation to the frequency of vertically orientated joints in the hangingwall, and pre-tensioning of the bolts may be required to induce friction between unbonded strata layers. If a fault plane is present in the hangingwall, the bolting pattern should be altered to support the loose wedge formed below the fault plane, and additional long reinforcement anchors should be installed if necessary.

Support requirements for *massive mining* methods vary with the type of mining and fall beyond the scope of this book.

The use of *backfill* as support has application in wide-reef mining. The main advantages of backfill are its good areal coverage properties, its ability to serve as a regional as well as local support medium, and its potential to permit increased percentage extraction in pillar mining systems. The stiffness of the backfill used is of crucial importance and, as a general rule, the stiffness requirement increases with increasing mining height and with decreasing closure rates. High initial stiffnesses can be achieved by the addition of up to 10 % cementitious binders to conventional tailings or aggregate mixes.

4.7.2 Multi-Reef Stope Support

The variables that influence the choice of stope support for multi-reef mining are similar to those considered in multi-reef layouts (Chapter 3.6.2). These include the mining depth, the distance between reefs, the effect of pillars and remnants, the influence of the surrounding rock type, and the effect of fracturing induced by previous mining. Thus, the support requirement for each multi-reef situation should be regarded as a unique problem with a unique solution.

When two reefs are mined, a **middling** is formed and it is important to assess the potential stability of this middling prior to the choice of stope support for either of the two reefs. At present, the determination of middling stability can best be achieved by a combination of back analysis and local knowledge. As a general rule, middling stability decreases with increasing mining span if total closure has not occurred on either of the reefs.

The mining heights of multi-reef stopes are subject to the same support limitations that apply to single reef mining, namely that, beyond a mining height of about 2 m, conventional pack or stick support systems become increasingly ineffective and difficult to install. In the range 2 - 3.5 m, backfill, cluster sticks or grout packs may be used, and beyond 3.5 m, pillars may be used in certain circumstances.

Two basic situations are encountered in multi-reef mining: the second and successive reefs are mined subsequent to the extraction of the first, or, two or more reefs are mined simultaneously. The considerations appropriate to these two situations are first reviewed below, and then individual guidelines for shallow, intermediate and deep conditions are presented.

When **subsequent** extraction is practiced, pillars and remnants will often be present on the first mined reef horizon and these must be taken into consideration when defining the support requirements in the second and subsequent reefs to be extracted; remnant precautions are typically required when mining within about 50 m of such structures. If total closure has occurred over much of the first mined horizon, stresses will be regenerated and may approach or even exceed the original virgin stress; from this point of view it may be desirable to record on stope planning sheets the stoping width used on the extracted horizon. Where total closure has not occurred on the first mined stopes (in shallow mining conditions, for example), the extraction of the second reef will occur in partially distressed conditions. This will limit the amount of closure that will occur during the extraction of this reef and for this reason *stiff* support systems (for example, backfill or yield pillars) should be used. Middling stability is always a major consideration. Finally, mining a second reef at depth can reactivate seismicity and therefore support may have to cater for the possibility of rockbursts.

When **simultaneous** extraction is practiced, important considerations include middling stability, the tailoring of support to the differing stress conditions expected on the two horizons, and the possibilities of seismic activity. The use of backfill greatly assists in addressing these problems. Unmined ground, such as pillars or remnants left on either of the reefs, can result in stress problems on the other stoping horizon, requiring additional layout and support precautions.

(i) Subsequent extraction (shallow mining conditions)

At shallow depths, complete stope closure is rare. If the reefs are sufficiently far apart, middling instability will not develop and the operation may be treated as the mining of two separate reefs for the purposes of support. At the other extreme (beyond span:middling ratios of about 3:1, but very dependent on local conditions), the stope support system of the lower reef has to ensure middling stability by having the capacity to bear the full deadweight of the middling strata.

If a lower reef is to be mined and middling instability is anticipated, stabilization can be achieved by the use of pillars which limit mining spans, or by means of stiff high-yield support systems such as backfill or sandwich packs. It is important that the full height of the unstable middling be supported.

If an upper reef is to be mined, re-support or prior backfilling of the lower stoped horizon should be considered in order to avoid sag of the middling and loss of sup-

port reaction in the upper workings. Roofbolting of the upper reef may also have to be resorted to in this particular scenario.

(ii) Subsequent extraction (intermediate depth conditions)

At intermediate depth, total stope closure can occur over large areas of the extracted reef horizon, particularly in longwall mining, leading to the regeneration of the virgin state of stress. If pillars, remnants, geological losses, localized waste filling or narrow stoping widths are present, stresses in excess of virgin stress can occur and may affect mining conditions adversely during the extraction of the second reef. This would take the form of excessive closure and stress-induced face fracturing, and possible significant seismic activity, and it is important that these factors be considered when selecting a suitable support system. In severe cases, the extraction of the second reef may warrant the use of remnant support precautions (section 4.6.1). If the stope middling is less than about 20 m, stoping on the second reef may intersect the stress fractures formed during the extraction of the first reef (see section 3.6.2). The density and orientation of these fractures should be considered when selecting a support system for the extraction of the second reef. If seismicity occurred during the extraction of the first reef, it is possible that the mining of the second reef can reactivate this seismicity; consequently, rockburst-resistant support systems may have to be used in places.

If a lower reef is to be mined, support considerations are similar to those in shallow mining conditions: the middling stability should be evaluated and if necessary the full height of the middling should be supported, using backfill, grout packs or sandwich packs. Stress-induced fracturing, formed during the extraction of the first reef, can be intersected during the extraction of the second reef. This necessitates the use of a support system having good areal support capability, particularly in the stope face area. If pillars are to be undermined, remnant precautions (section 4.6.1) will often have to be implemented.

If an upper reef is to be mined, severe difficulties can be expected if middling instability is a possibility. This is the case if the span:middling ratio is several times greater than unity and the lower reef is not totally closed but is unavailable for backfilling prior to the mining of the second reef. Instability will take the form of footwall sag with loss of support reaction of the upper reef support system. To counter this problem, use of high areal coverage support (roofbolting, high density packs) is advocated for the upper reef mining.

(iii) Subsequent extraction (deep mining conditions)

In deep mining conditions where total extraction was practiced, remnants and pillars are rare and total stope closure is common over large areas of the extracted reef horizon. Stress levels approaching the original virgin stress magnitudes are regenerated through these closed areas, and in these circumstances the extraction of the second reef will present problems no worse than those experienced during the extraction of the first reef, though in narrow-middling situations (<30 m), increased areal coverage support may be required to cope with stress fracturing associated with the first reef.

However, as in intermediate depth mining, if localised waste filling or local areas of narrow stoping widths were present on the first reef, stresses in excess of virgin stress

can occur and these may affect mining conditions adversely during the extraction of the second reef. Even more severe conditions will exist if pillars were left on the first mined reef; stress and seismicity levels in the vicinity of these pillars can be extremely high. If it is possible to extract portions of the second reef under these conditions, then rigorous support requirements will apply and the use of remnant support precautions (section 4.6.1) will normally be warranted.

If seismic activity was present during the extraction of the first reef, it will often be remobilized by subsequent mining activities. Support designs should therefore in general take the possibility of rockbursting into account.

(iv) Simultaneous extraction (shallow mining conditions)

Middling stability between the two reefs is a major consideration; if instability is anticipated then the support system on the lower reef must be capable of supporting the full deadweight involved. Closure in these stopes will be reduced due to the overmining or undermining environment and, as a result, *stiff* support should be used. For example, if the top reef leads the lower reef, then pillars are often the best support option for the lower reef, particularly if the mining width is high.

Three lead/lag strategies of simultaneous extraction are possible and are discussed individually below.

If the two reefs are extracted with approximately *in line* faces (neither leading the other), the support used must be stiff to cope with the low closure rates encountered; for example, yielding timber props, grout packs, sandwich packs or cemented backfill.

If the *upper face leads the lower*, the lower reef will be destressed and here attention must be paid to using particularly stiff support such as cemented backfill.

If the *lower face leads the upper*, backfill can be used on both reefs; and if the lower face leads by a large amount, closure over this backfilled horizon will partially negate the effects of destressing when the upper reef is mined (advantageous in terms of strata control).

In all cases, if a *middling stability* problem is anticipated, the support system in the lower reef horizon must be capable of supporting the full deadweight of the middling; this can generally be achieved by the use of in-stope pillars or backfill.

(v) Simultaneous extraction (intermediate depth conditions)

In intermediate depth conditions, it is desirable that one of the reefs should lead the other at a distance such that the abutment stresses from the leading reef do not affect the face of the trailing reef. In these circumstances the trailing reef will be destressed. Stress-induced fracturing will occur on the leading reef and the trailing reef can intersect these fractures if the middling between the reefs is small. Because the trailing reef is destressed, the blocks bounded by these fractures can be loose and the support in the stope face area of the trailing reef should supply good areal coverage. Blasting problems may also occur and special blasting precautions may have to be implemented.

If the *upper face leads the lower*, support in the lower reef must be stiff with a high areal support capability, particularly if stress-induced fractures are intersected from the upper reef mining. If this is the case, remote-release mechanical props or hydraulic props with suitable headboards may be used. Permanent support in the lower reef should be stiff and, if necessary, be capable of stabilizing the middling; backfill in conjunction with elongates will fulfil these roles. Support requirements are less rigorous on the upper reef and should be evaluated in terms of normal intermediate depth mining conditions.

If the *lower face leads the upper* by a substantial distance and backfill is placed in this horizon, a narrow middling will be stabilized ready for extraction of the upper reef. If backfill is not used in the lower reef and a middling stability problem is anticipated, the support in this reef must be capable of supporting the full deadweight of the middling; crush pillars are the best practical option under these conditions.

As a general rule, the principle of mining the upper reef's first ('**top-down**') offers very considerable advantages in terms of both safety and productivity; and this needs to be considered very seriously in the early stages of mine layout and design.

(vi) Simultaneous extraction (deep mining conditions)

Although no examples exist in practice, the considerations discussed for simultaneous extraction at intermediate depth should also apply to deep mining conditions.

4.7.3 Steep Slope Support

Several mining methods can be used for the extraction of steeply dipping reefs. These include variations of walled shrinkage and sub-level shrinkage with overhand, breast or underhand configurations. They mainly use mat or sandwich pack support in the face and back areas and along accessways; elongates are also used successfully. Use of face temporary support such as timber sprags or lightweight mechanical props is advocated; these should be orientated perpendicular to the reef and may be overlain with planks to form a drilling platform and 'umbrella' protection against falling objects. Heavy support units are themselves potentially hazardous and need to be provided with appropriate safety measures to prevent their falling down the slope, for example, tethering by means of chains.

In situations where the dip angle lies between 35° (the threshold of 'steep' mining) and about 50°, most of the support considerations of sections 4.2-4.4 continue to apply. For dips steeper than about 50°, friction alone is sufficient to render keyblocks in the hangingwall self-stable and thus falls of ground, in the normal sense of the word, are relatively infrequent even in the absence of conventional support. Under these conditions, shrinkage provides good permanent rockburst support. Consideration should also be given to the use of lightweight rapid-yield hydraulic props or timber elongates in the face area for control of potential rockburst damage. When backfill is intended to be placed concurrently with normal stoping operations, the dip of the reef and shear angle of the fill material should be considered, and adequate precautions should be taken to reduce the possibility of the fill material slumping into the downdip panels and gullies of the slope.

In gullies not protected by sidings, serious consideration should be given to providing tendon reinforcement to the gully hangingwall and down-dip sidewalls.

4.8 STOPE SUPPORT - QUALITY CONTROL

The implementation of a sound quality control programme, covering standards of installation and integrity of support elements used, is necessary to ensure the maximum safety and productivity of ongoing stoping operations.

The integrity of support elements may be checked by carrying out random sample testing (in the case of timber units, which are susceptible to drying-out, both on receipt and immediately prior to being sent underground), along the lines described in Chapter 6.5.3. Particular attention needs to be paid to hydraulic prop control; special procedures need to be set up for the efficient testing, maintenance and deployment of props. Similar requirements apply to backfill quality control.

The importance of standards covering making-safe procedures, as well as quality of installation of temporary and permanent stope support units, cannot be over-emphasized. These standards will vary considerably according to local conditions, but appropriate ongoing management attention to aspects of standard setting, training, discipline, and supervision will pay significant dividends in terms of enhanced safety and productivity achievements. The unpopular but necessary aspect of monitoring should also not be overlooked: the checking on a random basis that standards are being adhered to, that inappropriate short-cuts are not being followed, and that contingency support units are available and being used where necessary.

5.1 INTRODUCTION

The South African gold and platinum mining industry develops an astonishing 800 km of *new* tunnels annually, and approximately 10 000 km of tunnels are in use at any one time. These excavations form the vital link between the stoping excavations and the shaft systems, for transport of personnel and material, ventilation and the removal of ore. From a safety point of view their stability is important with regard to personnel utilising them, as well as for effective ventilation of the mine. The maintenance of operational tunnels through good design practice becomes crucial in the deep South African mining environment, where failure of the rockwalls can lead to poor ground conditions and unacceptably large deformations.

Analysis of accident data shows that approximately 16% of rock-related fatalities in the South African mining industry occur in tunnels. A high proportion of these fatalities occur less than 5 m back from the face, during development (Figure 1.2.7); however, a substantial number occur in operational tunnels where rockwall stability should have been established. Of the rock-related fatalities in tunnels, about 40% overall are associated with rockbursts and the balance with rockfalls. In the deeper mining districts, this proportion increases to approximately 50%.

The stability of tunnels is controlled by the efficacy of the support system in use (the subject of Chapter 6), but also to a considerable extent by the field stress and rock mass environment, and the interaction of the excavation with this rock mass. By appropriate design of the layout of tunnels (their size, shape, orientation and excavation technique) within a particular geotechnical environment (rock mass strength and stress), the intrinsic stability of tunnels can be maximised, and support and rehabilitation costs kept to a minimum. These layout design considerations are the topic of this chapter. The aspect of tunnel location associated with different mining layouts was discussed in Chapter 3.4: shallow mining access in 3.4.1(e), medium depth (scattered mining) access in 3.4.2(c), deep longwall access in 3.4.3(b) and deep 'sequential grid' (SMDP) access in 3.4.4.

5.2 ROCK MASS ENVIRONMENT

In most cases the stoping layout dictates the general location and orientation of tunnels, and only limited flexibility is available for modifying this location. Nevertheless tunnels should be sited in the most favourable rock mass environment

available, in order to minimise the degree of fracturing and/or rock mass deformation. This is determined by the relationship between the level of stresses acting on the tunnel site and the immediate rock mass strength characteristics. Furthermore, tunnels should avoid running parallel to seismically active structures at distances of less than about 50 m. Where necessary, they should traverse such dykes or faults (as well as any particularly weak or jointed formations) as close to perpendicular as possible, in order to minimize the length of tunnel exposed to the hazards involved.

5.2.1 Rock Mass Strength Characteristics

The **strength** of a rock mass is determined by the *intact rock strength*, and the disposition of *discontinuities* within the rock mass. The lower the intact rock strength and the greater the density of discontinuities, the weaker the rock mass strength becomes. For a given stress state, a low rock mass strength implies a greater potential for fracturing, rock mass deformation and excavation instability. An estimation of the rock mass behaviour may be determined by the application of rock mass classification techniques (Chapter 10.3.5). These provide a rational basis for the *downgrading* of intact rock strengths (established from small laboratory-scale testing) in terms of the number, spacing and properties of joint sets and other discontinuities in the rock mass.

In principal, tunnels should be sited in strata with the highest rock mass strength in order to maximise their stability in relation to the stress environment. However, certain mining methods allow only limited flexibility with regard to the positioning of tunnels and it may be necessary that excavations traverse zones with poor rock mass characteristics. The increased likelihood of fracturing and rock mass deformation may be accommodated by the correct design of support systems for these poorer anticipated conditions. However the length of tunnel which may be exposed to adverse rock mass characteristics should be minimized by traversing zones of weakness at high angles of incidence, i.e. close to perpendicular (Figure 5.2.1). In addition, reef extraction layouts should avoid the leaving of pillars or abutments that would expose off-reef excavations to elevated stress levels, particularly in poor rock mass conditions.

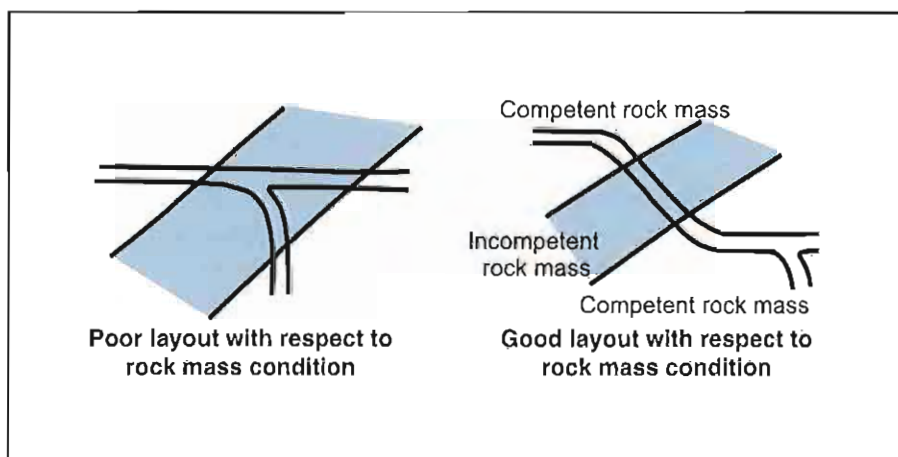


Figure 5.2.1. Considerations of tunnel layouts with regard to poor rock mass conditions.

Typical areas where special attention is required for the layout of tunnels include the negotiation of faults, fault zones and dykes, where a much more intensely jointed and altered rock mass may be encountered. Highly-laminated strata with a tendency for unravelling can also pose problems.

Tunnels in close proximity to the mining horizon may also be influenced by fracturing associated with the stoping, resulting in a more discontinuous rock mass structure. Follow-behind haulages in deep level longwall mining should be located not closer than 30 m below the reef in order to avoid interaction with stope fracturing. However to enable crosscuts to be broken away as close to the stope face as possible, such that boxholes will be operational reasonably close to the face, footwall drives are often positioned closer to the reef than 30 m. In this situation support design must cater for the increased blockiness of the strata. Crosscuts developed for access to the reef horizon will inevitably intersect fracturing running sub-parallel to the crosscut due to the stoping operations and thus may require additional support. These issues are discussed further in Chapter 3.4.

In areas of poor rock mass stability, the size of tunnels should be kept to a minimum and any layout complexities avoided, in order to reduce the problems likely to be encountered (Figure 5.2.1). For example, breakaways, double-width tunnels, loops and waiting places should not be sited in such conditions if at all possible.

In certain platinum mines, areas can be highly serpentinized. Jointing is intense and cohesion or friction across interfaces is low, with the result that the rock mass is highly discontinuous. Developing and supporting in such areas is particularly difficult, and even pre-grouting ahead of the face has not proven very effective. Prior geological exploration is of prime importance, in that development can be planned to minimise the length of tunnels which have to traverse such ground.

5.2.2 Stress Environment

Tunnels should ideally be located where the stress environment, both current and future, will result in minimal fracturing of the rock mass surrounding the excavation. A suitable criterion to apply for this purpose is the Rockwall Condition Factor (RCF) – Chapter 3.2.9. Visible stress-induced fracturing of the peripheral rock mass (in good-quality quartzitic environments) will typically initiate at an RCF of about 0.7 (i.e. an absolute maximum rockwall stress of about 0.7 times the uniaxial compressive strength of the intact rock), and will be oriented parallel to the maximum principal stress. The development of an excavation will result in stress concentrations on the boundaries of between 2 to 3 times the local field stress, dependent on the shape of the excavation and the k ratio (ratio of horizontal to vertical stress).

Where the maximum principal stress is vertical, stress-induced fracturing will occur predominantly in the sidewalls of the excavation – Figure 5.2.2. However, tunnels located close to abutments or in steeply-dipping orebodies (where the major principal stresses can be sub-horizontal) tend to exhibit fracturing in opposite corners or in the hanging and footwall of the excavation. Similarly, where high virgin horizontal stresses are experienced (in some platinum mines, for example), ground control problems may be associated with the hangingwall of the tunnels, and what is locally termed ‘Gothic-arch fracturing’ will occur.

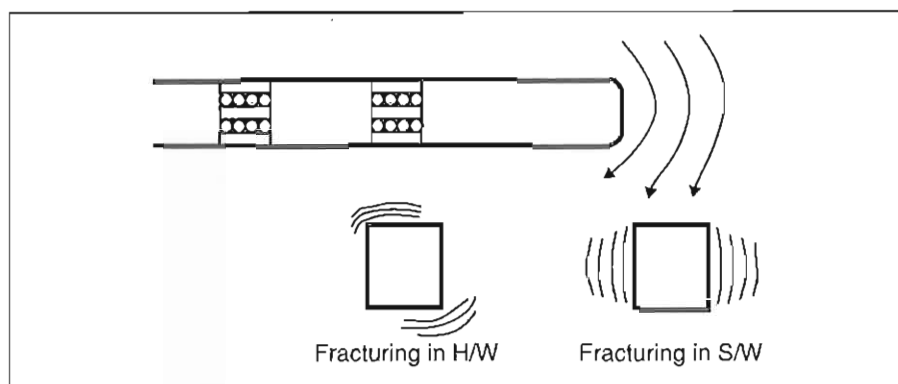


Figure 5.2.2. Typical rockwall fracturing and deformation in tunnels

These fracturing processes result in dilation of the surrounding rock mass and deformation of the tunnel rockwalls. A major objective of the layout of tunnels is to minimise these deformations so as to maximise excavation stability. Where **high field stresses** are anticipated, planning should aim to maximise the distance from stress-concentrating pillars and abutments, to keep the size of the excavations as small as practical, and to avoid complex geometries and interactions with other excavations, in order to maximise the integrity of the rock mass.

Many tunnels are subjected to **stress changes** over their operational life. A substantial stress increase may result in either the initiation of stress-induced fracturing, or further development and deformation of a pre-existing fracture zone around the tunnel, depending on the current state of the peripheral rock mass. Case studies have also indicated that a *reduction* in the stress field will often lead to further deformation, often in the hangingwall, of the rock mass around a tunnel. It is thought that this deformation is due to relaxing shear and dilatatory movements within the discontinuous rock mass surrounding the tunnel. Excavations should thus also be sited so as to *minimise the degree of stress change* to which they may be exposed.

Overstopping is an effective method of protecting off-reef excavations from the effects of large mining-induced stress changes. It should be noted however that where field **k-ratios are high** overstopping of an excavation may, dependent on the actual scenario, be *detrimental* to its stability (as has found to be the case in 'Gothic arch' situations).

Thus in designing tunnel layouts it is desirable to estimate the magnitude, and potential changes, of the stress fields due to mining. These may be determined with the aid of **numerical modelling** (Chapter 11.4), and evaluated in terms of the selected **criteria** for excavation stability or support requirements (Chapter 6).

5.3 EXCAVATION SIZE AND SHAPE

In general the size and shape of a mining tunnel are determined by operational considerations such as the equipment to be moved through the excavation; the require-

ment to carry services such as water and compressed air; and the important requirement for transmission of sufficient ventilating air. However, particularly in problematic or high-stress ground conditions, consideration must also be given to rock mechanics design principles with regard to the size and shape of the excavations.

5.3.1 Excavation Shape

The shape of a tunnel determines to a considerable extent the magnitude of the induced stress levels on the rockwalls, and thus the potential for stress-induced fracturing. In order to minimise the induced compressive stresses on the periphery of an excavation, the theoretical optimum shape is an ellipse with the long axis parallel to the major stress direction (Figure 5.3.1). This practice, which can help prevent any sidewall fracturing in low-stress environments, is commonly utilised in large civil engineering chambers. It is rarely used in deep mining applications due to (i) excavation difficulties; (ii) the important fact that in high-stress environments where the major stress is near vertical, heavy fracturing is inevitable and in a high sidewall, is in fact dangerous. Paradoxically, under these conditions, the most stable shape is the reverse of the above: stability is maximised for an ellipse with long axis *normal* to the major stress direction. High-stress tunnels should be designed with this type of elongation in mind.

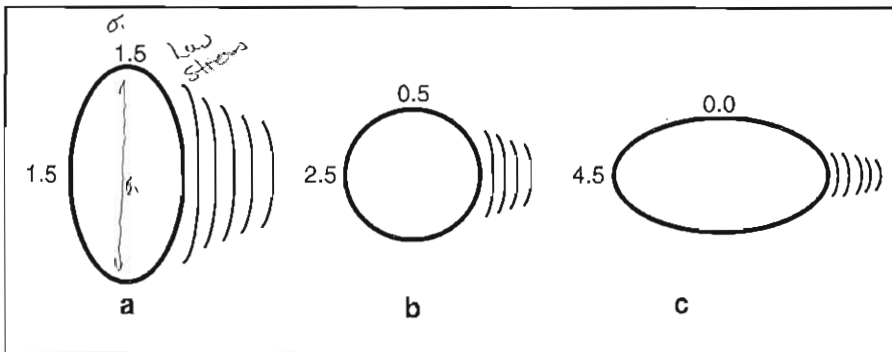


Figure 5.3.1 Influence of tunnel shape on stress levels and extent of fracture development. Stress concentrations (multiples of vertical field stress q_v) are indicated on the periphery of tunnels in an *elastic* (low-stress) environment, for a k -ratio of 0.5. In a high-stress environment where the sidewalls can fail, relative depths of fracturing are illustrated.

The geological structure of the rock mass being traversed – the bedding and jointing orientations – also play a role in determining the most appropriate cross-sectional shape of a mining tunnel. In *low stress environments* the appropriate shape may be fully determined by a dominant geological structure within the rock mass; e.g. the development of a 'lean-to' profile in highly bedded strata. The profile then conforms to the dominant structure and minimises the potential for the creation of isolated unstable wedges in the immediate hangingwall. In highly-jointed rock masses, the adoption of an excavation profile already conforming to the 'naturally stable' shape may similarly improve safety and minimise support requirements (Figure 5.3.2).

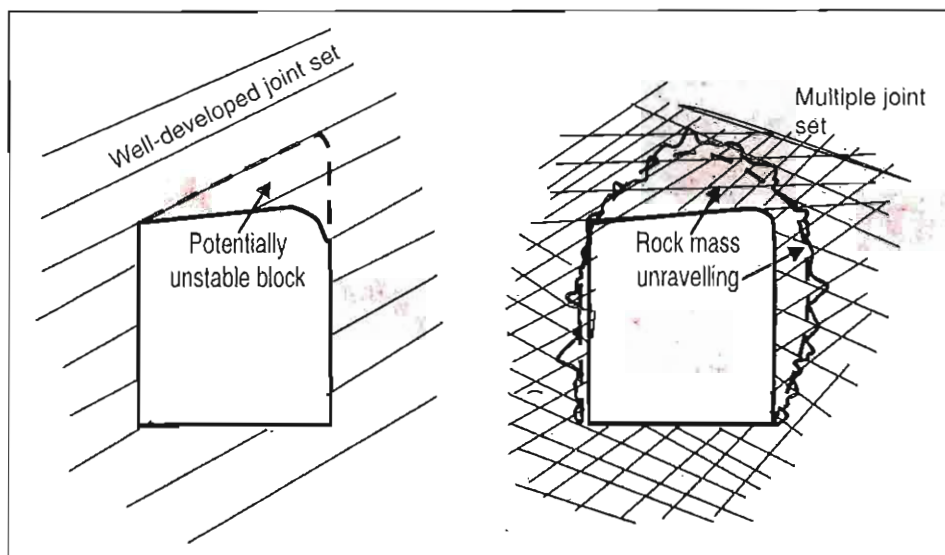


Figure 5.3.2 Use of tunnel shape to minimise instabilities within the surrounding rock mass (dashed line indicates preferred blasted tunnel outlines)

Where the stress environment is such that extensive stress-induced fracturing around the tunnel cannot be avoided, mining practice has shown that an elliptical shaped excavation, with the long axis normal to the maximum principal stress direction, will minimise the extent of the fracturing (Figure 5.3.1c). This shape allows rapid generation of confining stresses and thus triaxial stability of the rock at a short distance into the rockwall of the excavation. In addition the small height of the rockwall parallel to the maximum principal stress is considered to result in a far more competent rock mass structure that limits deformation and thus provides resistance to further development of the fracture zone, as well as effectively reducing sidewall support requirements.

An elliptical shape may be difficult to excavate in practice (though an equivalent vertically oriented profile is utilised in the platinum mines of the Bushveld Complex where 'Gothic arch' tunnels are developed in areas of high horizontal stress). In general, an effort should be made to reduce the radius of curvature of the rockwall parallel to the maximum principal stress direction if problems are envisaged with the stability of an excavation in a high stress environment. Although this shape may be advantageous, it should be born in mind that the creation of a large span in the minimum principal stress direction can pose inherent problems if the excavation is to be subsequently overstoped and stress-relieved. Thus ideally, the influence of the full stress history of a given excavation needs to be considered prior to its development.

5.3.2 Excavation Size

The *size* of an excavation does not directly influence the magnitude of the induced stresses, as these are purely a function of the shape. However, the size of an excavation does influence the volume of exposed rock and associated discontinuities, thus potentially influencing the effective strength of the peripheral rock mass. The size

will also determine the depth into the rock mass which is subjected to a given elevated stress state. Thus a tunnel twice the diameter of a comparable tunnel, in the same rock mass environment, will experience potential stress-induced fracturing to twice the depth. This will result in correspondingly larger deformations of the tunnel rockwall (Figure 5.3.3). Consideration of this increased deformation is important in the assessment of the required support capacity and rock mass stability limits.

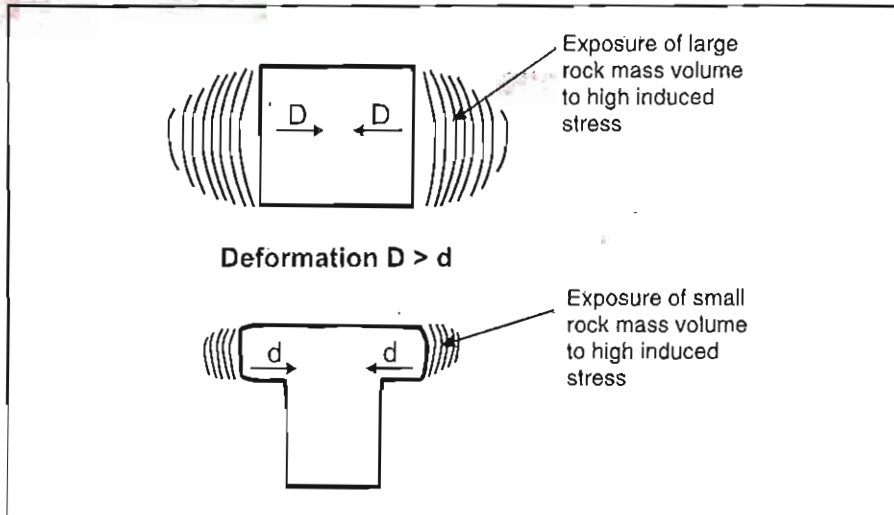


Figure 5.3.3. Influence of excavation size on extent of fracturing and rock mass stability.

In addition, the size of an excavation may limit the ability of the support (unless longer reinforcing tendons are used) to interact with the rock mass to create a competent reinforced rock mass shell (section 6.4.1.3); and thus the ability to restrict rock mass deformation to within stable and operational limits.

An example of the influence of excavation size and alternative excavation profiles is furnished by the development of reef raises in high-stress environments. This is often problematic due to the poor rock mass conditions on the reef horizon and the temporary nature of the support installed. The use of wide raising is thus sometimes practised under these conditions, involving the mining of a reef slot in advance of the raise or centre gully excavation. The principle of this methodology is to expose a small sidewall profile to the high abutment stresses and thus maximise the face stability due to its limited height (Figure 5.3.3), while the main access development, comprising the centre gully excavation, is developed in de-stressed ground and the fractures developed ahead of the slot will intersect the gully at high angles, thus increasing gully shoulder stability. However a disadvantage of this scheme is the development of shallow dipping fractures over the heading which then requires additional, preferably tendon support. These comments also apply to 'T' shaped tunnels which have been tried for flat development and shown to be viable under very high stress conditions.

As a general rule, the size of any service excavation should be made as small as pos-

sible for the anticipated operational requirements. This is particularly true in adverse rock mass environments, but due allowance must be made for the inevitable dilatational deformations that will occur over a period of time.

5.4 EXCAVATION POSITIONING

The positioning of any excavation relative to external factors can play an important role in the behaviour of the peripheral rock mass, and ideally a layout design should place the excavation in the most favourable conditions. In many instances however, even though adverse rock mass conditions are anticipated, the layout of a given tunnel may be fixed by external considerations such as infrastructure requirements or simple mining economics. Under such conditions, upgraded support systems need to be implemented in order to accommodate and control the anticipated additional rock mass deformations so as to maintain excavation stability and operational safety.

Given more freedom of action, factors which need to be considered include the orientation of tunnels with respect to geotechnical structures within the rock mass, and the major principal stress direction. Tunnels may also be influenced by adjacent excavations, either other off-reef service excavations or stoping. Consideration must also be taken of the risk of exposure to seismicity associated with geological structures or major stoping abutments. These factors are discussed in the following sections.

5.4.1 Orientation

The layout of a tunnel or service excavation may have some flexibility with regard to orientation, either over its whole length such as a major chamber, or over a portion of its length such as a tunnel negotiating a hazardous rock mass environment. The principle consideration of excavation orientation is in regard to a dominant, and potentially unstable, geological structure within the rock mass. This may be a ubiquitous structure such as a weak joint set, or an isolated structure such as a fault or dyke. Under these conditions it is recommended that the long axis of the excavation be oriented *perpendicular to the strike of the dominant plane(s) of weakness*. This orientation limits the length of intersection with potentially unstable structures created by the dominant planes of weakness, and thus maximises the stability of the excavation. However as the most common orientations of development in mines are, either on strike (drives) or on dip (crosscuts), one or other of these will be exposed to a less favourable orientation of the geological structure. This needs to be taken into account in the design of support.

In certain adverse environments in which a damagingly high sub-horizontal field stress operates, it may be desirable to adjust the orientation of an excavation in order to minimise the degree of resulting stress fracturing. In these environments it is recommended that the tunnel be oriented *parallel to the major horizontal stress direction*, where possible.

→ An extreme example of the influence of the orientation of the virgin stress field is the design of crosscut profiles in some of the platinum mines of the Bushveld Complex. Here the high horizontal stresses oriented along the strike of the orebody result in poor hangingwall conditions, and main crosscuts from the shaft infrastructure to the footwall haulages servicing the reef horizon are pre-developed with a 'Gothic arch'

profile. Ordinary crosscuts to the reef horizon are not used because of these problems, but inclined travellingways and orepasses are developed as a viable alternative. The advantage of not utilising ordinary crosscuts is both to minimise the length of excavation oriented perpendicular to the high horizontal stress field, and also to minimise the size of the excavations to access the reef horizon (Figure 5.4.1).

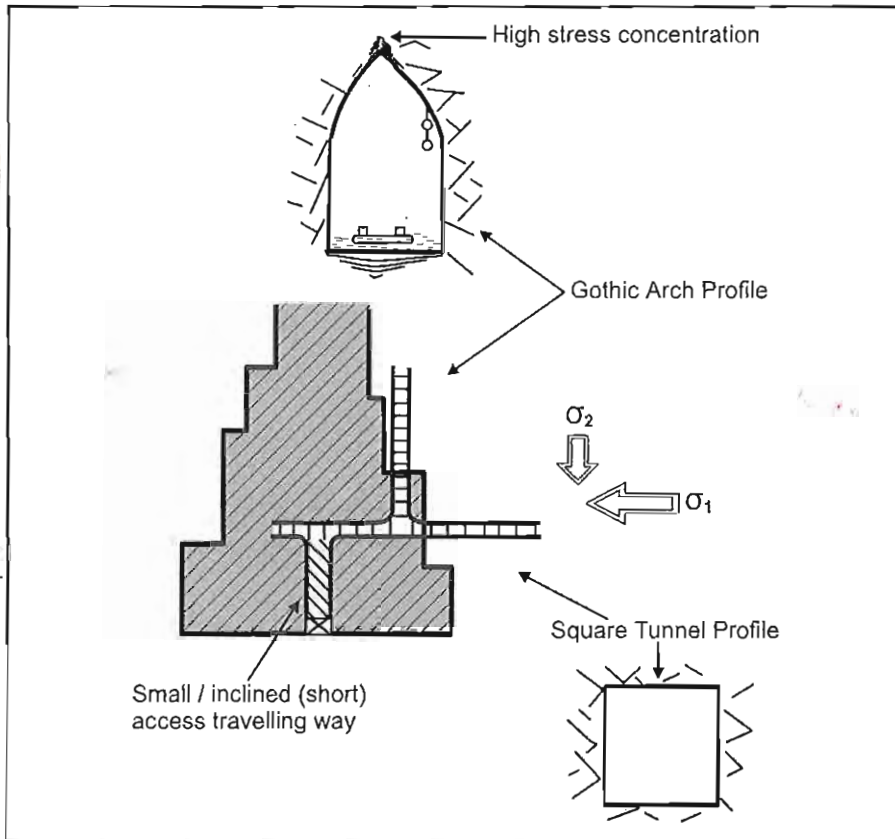


Figure 5.4.1 Example of tunnel layout in a high horizontal stress environment.

A second example of excavation orientation relative to the stress environment is the use of *vertical* dam and settler chambers at the bottom of deep shaft systems. With the vertical stress field being approximately twice that of the horizontal stresses, the fracturing around the excavations is primarily a function of the two sub-horizontal stresses which are normally of significantly lower magnitude.

5.4.2 Excavation Interaction

Any excavation generates a localised stress concentration around its boundary, which (in a low-stress, elastic environment) has a peak immediately adjacent to the excavation and decays to the field stress value at a distance of approximately two diameters from the centreline of the excavation. As discussed previously the degree of stress concentration is a function of the shape of the excavation. In addition, the peak stress concentration, and thus the profile of induced stress decay, can be shifted out from the

immediate excavation rockwall due to fracturing around the excavation (Figure 5.4.2).

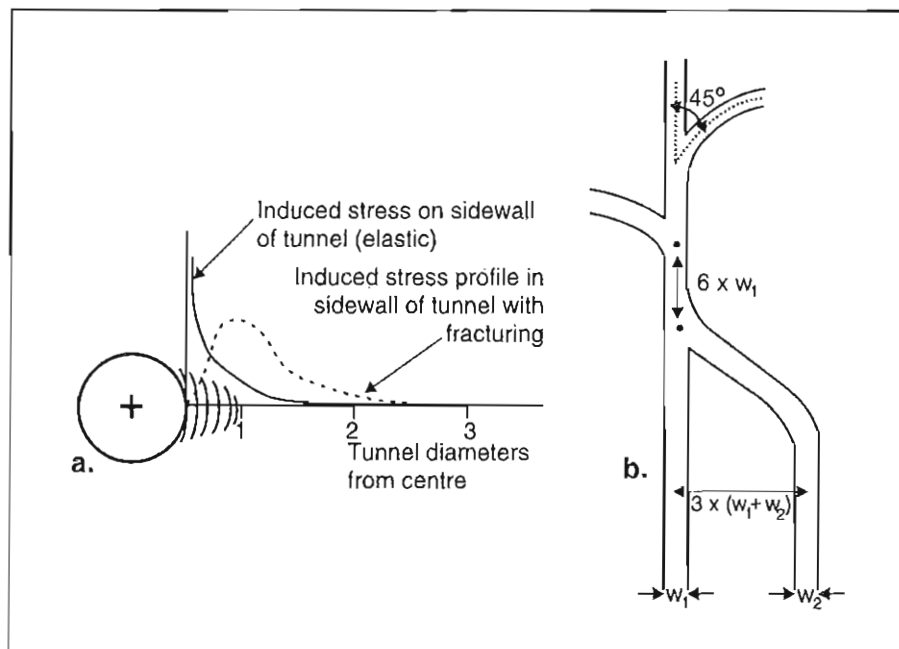


Figure 5.4.2 a: Induced stress profiles b: positioning of tunnels to avoid stress interaction (high-stress conditions).

Even in a low-stress ($RCF < 0.7$) environment, to avoid deleterious **interaction** between adjacent excavations, they should be sited (centre-to-centre) no closer than twice the sum of their diameters apart. However, when the RCF is larger than about 1.0, or if large seismic events are a possibility, the excavations should be positioned (centre-to-centre) not closer than *three times the sum of their diameters apart* to take into account the effects of fracturing of the excavation rockwalls. For particularly sensitive excavations, the necessary spacings may best be determined with the aid of non-linear numerical modelling codes (Chapter 11.4). If practical considerations do not allow sufficient separation, then interactions must be taken into consideration with regard to the potential for increased deformation and the requirement for additional support installation.

Where one tunnel is broken away from another, a pillar or 'bullnose', is created between the two excavations. This results in significant induced stress interaction and frequent failure of this pillar in the immediate breakaway area. In order to minimise the extent of this failure, the angle of breakaway should be as large as possible. The general design rule is that this angle should not be less than 45° . Again, if layout or equipment limitations do not permit this, additional bullnose failure must be anticipated with an associated increase in support demand. The interaction of excavations around a breakaway will also create a general increase in induced stress levels. In layouts where several breakaways are required in series, such as in shaft station layouts, these breakaways should be sited not closer than *six times the tunnel*

diameter apart in order to prevent increased interaction (Figure 5.4.2).

Excavation interaction should be assessed with due regard to the relative magnitude of the field stresses acting within the plane of analysis. Thus where the horizontal stresses are say approximately half the vertical stress level, the induced stresses and level of interaction of a vertically-stacked set of excavations would be more than proportionally reduced, thus allowing closer proximity of adjacent excavations in this plane for comparable levels of rock mass loading. Complex layouts, in general, require analysis by numerical models to assess the relative interaction of the excavations.

The design rules indicated above are based on the prevention of interaction of the induced stress fields between excavations. In some layouts it may not be possible to comply with these recommendations, in which case it is important to give consideration to the requirement for upgrading of the support in order to maintain the relative stability of the excavations involved.

Tunnels in close proximity to stoping excavations should be sited so as to minimise their exposure to abutment stresses, with regard to both stress increase and possible subsequent reduction. Thus major development to access stoping blocks should be sited sufficiently far from the orebody so as to maintain their long-term stability by minimising the potential for damaging stress changes associated with the stoping operations. Tunnels located too close to pillars or large-span abutments on the stoping horizon will experience high stresses and stress changes, and will experience correspondingly high deformations which will be difficult or impossible to contain. Numerical modelling and use of criteria such as RCF should be used to avoid such situations.

5.4.3 Seismic Hazards

The location of excavations in close proximity to areas of potential seismic hazard should be avoided, so as to minimise the potential for rockburst damage. Areas of potential risk need to be pre-identified by means of appropriate analyses of seismic data and by other geophysical means (Chapter 9). Areas of elevated risk typically include zones running parallel to and less than 30 m – 50 m from seismically-active geological structures such as faults and dykes, and major stoping abutments such as pillars and longwall stope faces. How far tunnels should be sited from such features should be based on the ability of the support system to maintain excavation stability under the anticipated seismic loading and on past experience in similar ground conditions. These guidelines should be periodically reviewed with changes in support and design technology.

If a seismically hazardous area has to be negotiated, the excavation should be oriented to expose the minimum length to the potential hazard and the support system needs to be upgraded, particularly in poor ground conditions. Typical support systems may include the use of long anchors in addition to the standard yielding rock bolts with mesh and lacing. Consideration should also be given to the reinforcement of the lower sidewall and footwall of the tunnel as these have been shown to be areas of weakness in current support system designs.

5.5 EXCAVATION TECHNIQUE

The method and sequence of development will influence the relative stability of the surrounding rock mass dependent on sensitivity to stress-induced fracturing and disruption due to the excavation process. Examples of these mechanisms, particularly visible in large chambers developed by sequential extraction, include the orientation of stress fracturing relative to the excavation profile and associated deformation history. The fracturing and damage to the excavation periphery due to blasting practice; or the sensitivity of the rock mass to exposure to water associated with the excavation process, can be important issues. The aspects of excavation sequence are covered under design considerations for large excavations (Chapter 7.2), and the following section is thus concerned primarily with excavation techniques associated with blasting practice in tunnels.

5.5.1 Blasting Practice

The action of blasting generates and propagates fractures within the rock mass as a function of the initial shock energy, subsequent gas emission and the ambient stress environment. Thus the damage to the rock mass will be a function of the rock mass characteristics and the blasting design. Mechanical excavation techniques generally result in minimal damage to the surrounding rock mass and thus rock mass damage is limited to stress-induced fracturing.

The direction of propagation of fracturing due to blasting will be controlled by the local stress environment. As with sidewall-parallel stress fracturing, blast-generated fractures will preferentially propagate in the direction of the maximum principal stress (Figure 5.5.1), the distance being determined by the energy available to continue the fracture propagation process. Energy is lost by creation of fresh fracture surfaces, or by escape of explosive gases along parting planes or fractures which intersect a free surface. Thus the degree of blast damage may be limited by either major partings within the rock mass or by the parameters of the blast design itself.

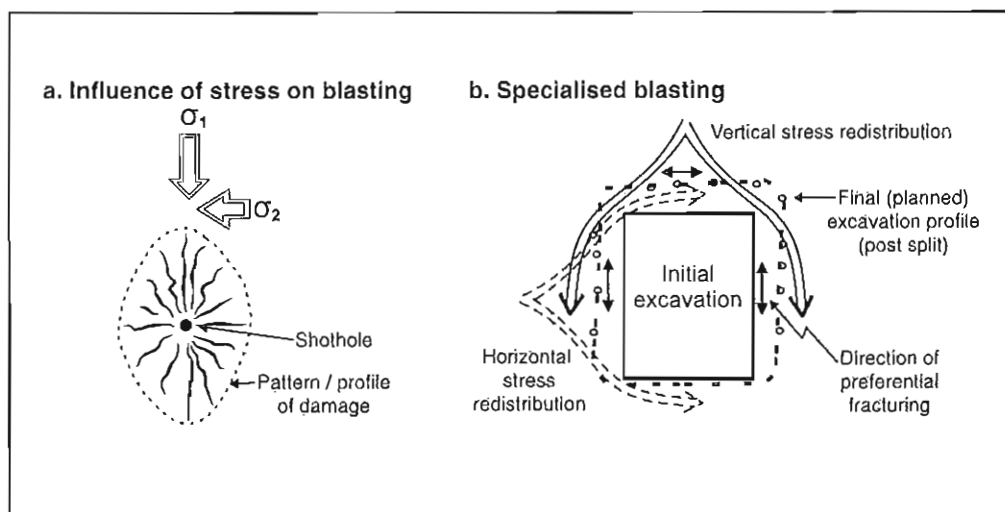


Figure 5.5.1. Considerations of blast design in tunnels taking into account orientation of field stresses

The most important parameter of the blast design is the **burden** of the shot holes, which is controlled by the hole spacing, the firing sequence, and the occurrence of misfires. As the burden of the holes increases, particularly in the direction perpendicular to the principal stress direction, so the depth of damage away from the excavation into the surrounding rock mass increases.

The principle of specialised blasting techniques is to take advantage of re-adjustments in the localised stress field in order to propagate blasting fractures in directions which enhance the stability of the excavation. **Post-splitting** involves the extraction of the central portion of an excavation prior to creating the final excavation profile; by these means the induced stresses on the periphery of the excavation are parallel to the rockwall (Figure 5.5.1). The blasting fractures generated by the post-split blast will thus preferentially form parallel to the boundary of the excavation creating less damage to the final excavation peripheral rock mass. Smooth wall blasting is known to reduce the combined effect of field stress and blast-induced fracturing in the rock peripheral to excavations. This is confirmed by significantly improved rock conditions around a bored raise compared to adjacent raises developed by conventional blasting. *Serious consideration should therefore be given to introducing smooth wall blasting* in situations where rock fracturing leads to excessively poor ground conditions which are difficult and expensive to support. In ultra-deep tunnelling situations, smooth wall blasting would provide significant benefits, and could well become essential in weaker rock horizons.

The blasting of unconventional tunnel profiles, such as elliptical or 'lean-to' shapes, may require some initial input from blasting experts as well as monitoring of early blasted profiles.

In general, the correct blasting techniques will improve both the stability of the resulting excavations and the effectiveness and cost of the support systems subsequently required.

6.1 INTRODUCTION

Ideally, the intrinsic stability of any service excavation should be maximised by suitable selection of its location, orientation and development profile (Chapter 5), so as to minimise the demand on the support system required. However, this is not always practical and support systems have to be designed for the prevailing conditions. High quality support needs to be installed wherever hazardous conditions exist, even if these extend for no more than a few metres along the tunnel.

Tunnel support systems may be made up of **reinforcing elements**, (such as end-anchored rock bolts, grouted tendons, Split Sets or injected grout) that act directly with the rock mass to increase its inherent strength; and **support elements, fabric support or coatings** (such as steel sets, mesh and lacing or shotcrete) which act to contain the inherently unstable rock mass between the reinforcing units.

In Chapter 5, the stability of a tunnel was evaluated with regard to its optimal siting, geometry and development. In conjunction with such strategies, support systems comprised of support elements of appropriate spacing, length and type provide protection against the hazards of rockfalls and rockbursts.

The fundamental purpose of tunnel support is to maintain the integrity and stability of the rockwalls under static and possibly dynamic (seismic) states of stress. In the first instance, the support system must control slow stress-driven deformations of the rock mass, which could otherwise disrupt the normal functions of the tunnel over its operational lifetime. A correctly chosen support system accomplishes these functions not only by supporting dead-weight, but by increasing inter-block frictional forces and resisting buckling of rockwall slabs; thereby constraining movement of key blocks in the rockwall and preventing unravelling and differential movement of the rock mass. In the case of dynamic loading the support system must absorb the kinetic energy imparted to the wall rock while maintaining the integrity of the discontinuous rock structure.

The sequence, and type, of support installation needs to be evaluated in relation to the anticipated deformation of the tunnel, thus taking into account different rates of deformation and the maximum expected deformation. A large percentage of accidents associated with tunnels occur within 10 m of the development face - See figure 1.2.7. Thus, high standards of **temporary support**, such as mechanical props with

canopies, need to be installed during the development stages. Where extremely poor ground conditions occur in the immediate face area, consideration should be given to the immediate application of high areal coverage support systems such as shotcrete or other sprayable material which can form a continuous membrane of sufficient strength. Temporary support is used to maintain safety in the face area during the early stages of the development cycle, including the installation of **primary support** tendons. This primary support is intended to maintain adequate protection to the development crew during further operations. The temporary support should only be removed once the primary support system has become fully effective. The design of the primary support should consider the factors detailed in the following sections.

To enable rapid development progress, the **primary support** needs to be designed only to cater for the current, short term, rock mass conditions. This will usually have to be upgraded with the installation of a **secondary support system** to cater for the longer term rock mass environment (stress changes, seismicity, time dependent deformations, etc.). At this stage, the function of the primary support is sometimes discounted; but, where appropriate, the contribution of the primary support can cost-effectively be integrated with the secondary support to comprise the final support system.

6.2 SUPPORT SYSTEM CHARACTERISTICS

Support units may be classified as being either active or passive types. **Active** (pre-tensioned) support types immediately apply forces resisting rock mass deformation, whereas **passive** support types require rock mass deformation to take place prior to the support system developing significant resistive loads. In general, it is advantageous to minimise the degree of rock mass deformation and thus maximise its inherent strength by the use of active support systems. However, the support system must also be able to accommodate the anticipated future rock mass deformation (yield) and associated loading of the support units.

A combination of (usually) several different reinforcing units and support element types, acting in concert with one another, comprises a **support system**, in which the contributions of the individual units may be appropriately added to give the overall support system characteristics.

In this book, the term **tendon** is used as a generic term to describe all reinforcing bars and cables; such as rock studs (often colloquially called rock bolts or roof bolts), grouted shepherds crooks or rebars, cable anchors, Split Sets, etc. The terminology for tunnel support is very inconsistent, not only in South Africa but world wide. As a consequence, the ISRM (International Society for Rock Mechanics) has established a committee to formulate a standard terminology to be used internationally in the future. The term 'rock stud' will probably disappear, and thus the common term 'rock bolt' will be used in this text. [Further details of support system terminology can be found in the '**glossary of terms**' at the end of this book.]

Key support characteristics are summarised in Figure 6.2.1, and are discussed further below.

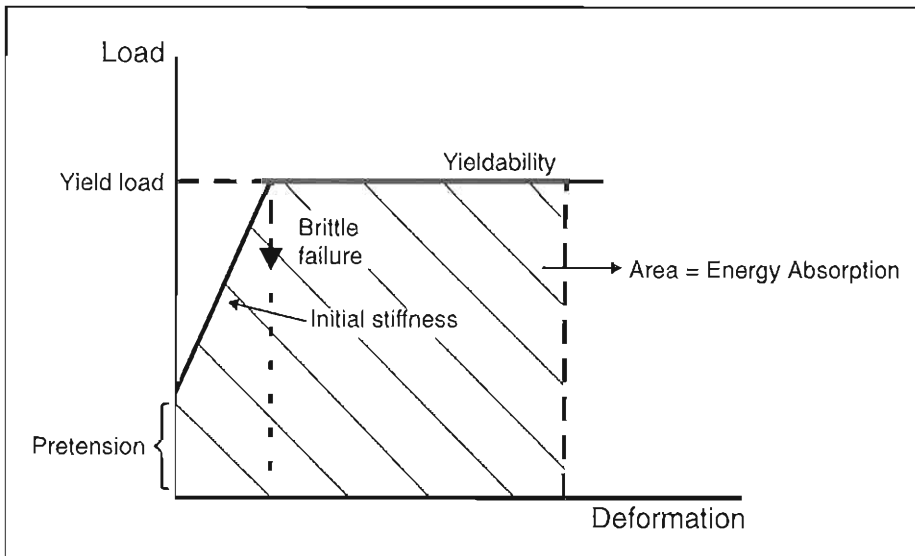


Figure 6.2.1. Key support characteristics.

6.2.1 Initial Stiffness

The action of rock mass reinforcement is to stabilise potentially unstable blocks and thus maintain the integrity of the overall rock mass structure. The **initial stiffness** is the rate at which load is developed within the support system with deformation (of the rock mass). In general, a high system stiffness is desirable. This is particularly true in low-stress environments, where activating deformations are small and where the stability of the rock mass surrounding the excavations tends to be controlled by geological structures. Pre-tensioning of rock bolts and other support elements greatly improves the ability of the support system to immediately do work against rock mass deformation. In higher stress environments where significant rock mass deformation occurs, the stiffness of the support system (usually of limited yield capability) must be compatible with the amount of rock mass deformation to be expected.

Examples of support elements with high initial stiffness include grouted bars and cables, pre-tensioned rock bolts, and shotcrete. Low initial stiffness systems include steel sets or arches with timber cribbing, and mesh and lacing areal coverage support.

6.2.2 Yieldability

Where significant rock mass deformation is anticipated due to a high stress environment, large stress changes or dynamic loading, the support system must be **yieldable** enough to accommodate this deformation. The yieldability of a support element is the total amount of deformation (normally well beyond the design peak load level) that the support can undergo prior to total failure. This yieldability can be expressed simply in units of mm, and also in terms of energy-absorption kJ units (see below). Interestingly, it has been shown that the shear capacity of a tendon is improved with increased axial yieldability.

Special yielding tendons are available, which have greatly enhanced yieldability compared to conventional tendons and which therefore stand up far better to seismic and other large-deformation loading. Cable anchors have also been shown to accommodate large deformations.

Yieldability is an important consideration for the elements of a support system which provide areal coverage (e.g. mesh and lace); but this must be considered in relation to the acceptability of differential deformations between the tendons and the areal coverage support.

6.2.3 Energy Absorbing Capacity

The yield capability of a support element in conjunction with its yield load will essentially define the amount of **energy** the support can absorb, particularly under dynamic loading conditions. This must be evaluated in relation to the volume and mechanism of dynamic loading of the potentially unstable rock mass with which the support element interacts.

It is commonly the case that the rock deformation *between* tendons in highly fractured tunnels is substantially greater than that immediately at the tendon (causing bulging of the mesh). The explanation is that there is limited interaction between tendons, with the result that the fabric support becomes overloaded. A further implication is that the tendons are imperfectly performing their function of rock reinforcement, and are not subjected to the loads and stresses of the full tributary area as is generally assumed. This also explains why a proportion of tendons with limited yieldability survive in rockburst conditions, and this fact is often used to justify not always using yielding tendons in rockburst prone areas. Where support systems are designed to ensure effective interaction between adjacent tendons, these will be subjected to most of the load or energy associated with its tributary area. Under such conditions, use of yielding tendons is essential - see section 6.4 on support design.

6.2.4 Length of Tendon

The **length** of tendon will determine the extent of interaction with the rock mass (Figure 6.2.2). In moderately stressed tunnels, support design philosophies in the South African mining environment are based on the concept of *anchoring* the potentially unstable rock mass around an excavation to the deeper, more competent, stable rock mass. The length of the tendon must be sufficient to accommodate the depth of instability and provide suitable anchorage in excess of this depth. The depth of potential instability may be derived from analysis of tunnel failure data or empirical guidelines. Empirical guidelines are based on an evaluation of the geotechnical structure of the rock mass for low-stress environments, and for high-stress environments on an analysis of the depth and stability of the fractured rock mass.

If the depth of rock mass instability is estimated to be greater than the practical tendon length, then the length of tendon to be used should be evaluated on the basis of

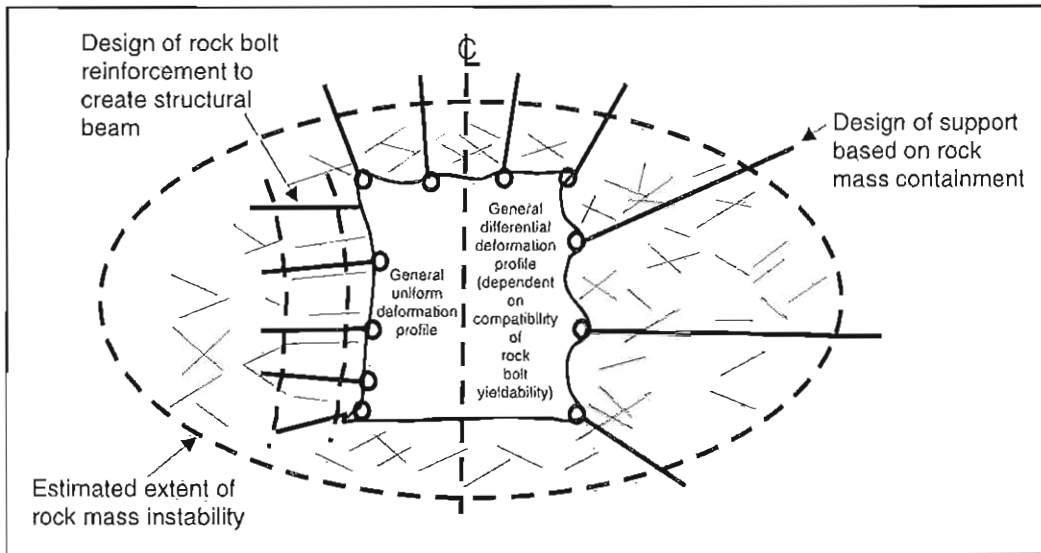


Figure 6.2.2 Illustration of two design concepts. Right - anchoring of unstable peripheral rock to competent rock mass. Left - creation of a structurally stable shell.

the creation of a *reinforced rock mass shell* of adequate shape and dimensions. Design methodologies are under development which attempt to evaluate the strength of this reinforced structure against the anticipated loading conditions. The methodology described in section 6.4 calculates the extent of interaction of the rock tendon reinforcements within the rock mass by taking into account the orientation and spacing of discontinuities in the rock mass.

6.2.5 Spacing of Tendons

The **spacing** of tendons within a support system will determine the loading of the tendons as a function of their interaction with the rock mass. Under static loading conditions, the loading of the support system may be expressed in terms of its *support resistance requirement*. Under dynamic loading conditions, this is best expressed as the *energy absorption requirement* of the support system. The loading of individual tendon elements will be a function of the spacing of the tendons and the depth of anticipated rock mass instability.

In highly discontinuous rock mass structures, it has been observed that the spacing of the tendons will also strongly influence the degree of loading of any areal coverage support between the tendons (Figure 6.2.2). Design methodologies need to consider the influence of the rock mass characteristics and the loading environment (static or dynamic) on the interaction of the tendons within the rock mass, and thus the necessity for areal coverage support requirements.

6.2.6 Areal Coverage

In order to maintain the overall integrity of the excavation, the potentially unstable rock mass between the tendons needs to be confined by areal coverage support sys-

tems (Figure 6.2.3). The requirement for **areal coverage** support is dependent on the stability of the rock mass between the tendons and is different for static and dynamic loading conditions. As the rock mass becomes more discontinuous, or the dynamic loading within the rock mass becomes greater, the direct interaction between the tendons and the rock mass is reduced. Thus the capacity and load-deformation requirement of the areal coverage support will be determined by the loading conditions on the rock mass, the extent of instability between tendons, and the allowable deformation limits of the excavation rockwalls.

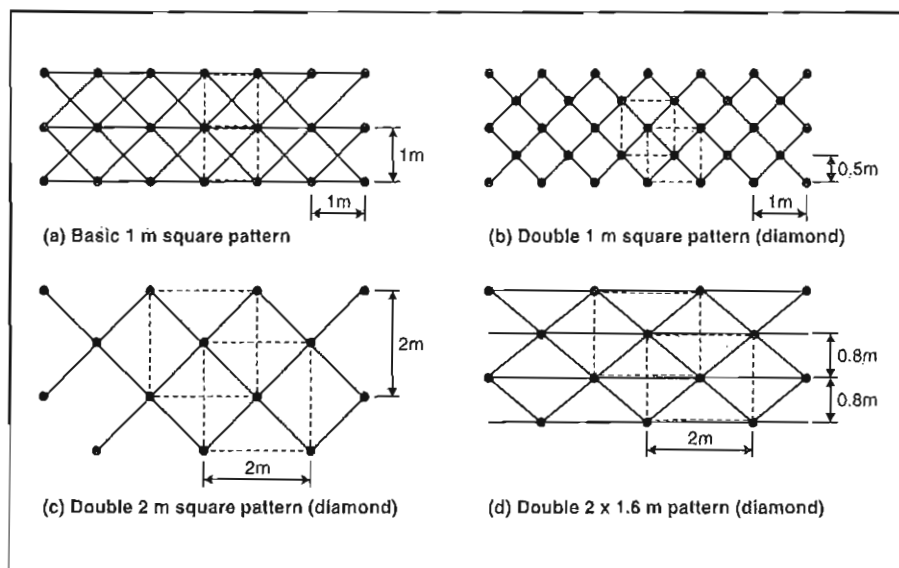


Figure 6.2.3 Typical tendon reinforcement and lacing patterns, giving different degrees of support resistance and areal coverage (mesh present but not illustrated). Dashed lines indicate basis of terminology: basic pattern (non-overlapping), double/diamond pattern. Lacing patterns are not restricted to those illustrated.

The design of a support system involves the matching of the strength, yieldability, length and particularly spacing of tendons to the yieldability, strength and stiffness of the fabric support for each geotechnical condition. In the past, little attention has been paid to the engineering design of fabric support, with the result that some sections of tunnels are in fact probably over-supported, while others may be under-supported with compromised safety.

6.2.7 Typical Support Element Design Characteristics

Table 6.2.1 indicates typical design characteristics of support elements utilised in the South African mining industry. However, it should be noted that the performance of a support element within a support system is also highly dependent on the interaction between the support element and the rock mass. An example is the often-observed survival of rebar tendons within a mesh and lacing support system after significant rockburst damage in highly stressed tunnels. This is due to the low degree of direct

interaction between the tendons and the rock mass, caused by the highly discontinuous nature of the rock mass. The large displacement of the rock mass and associated deformation of the relatively low stiffness mesh and lacing system either results in the absorption of the dynamic energy and stabilisation of the excavation without significant loading of the tendon anchorage, or failure of the mesh, if of insufficient capacity, and spillage of the fractured rock mass prior to any significant loading of the tendons.

Table 6.2.1 Support element design characteristics

Support element	Initial stiffness	Yield capacity	Load capacity	Shear capacity	Comments (applicability)
Friction tendon: Split Set/Swellex	fair	fair	low	fair/poor	Simple installation, corrosion a problem, Primary support only.
End Anchored pre-tensioned: Rock stud	v good	poor	med.	fair	Used under low stress conditions. Cables used for large excavations.
Cable anchor	v good	fair	high	good	
Fully grouted					
Smooth bar	good	fair	med.	fair	Easily debonded, requires good grouting.
Rebar	v good	poor	med.	poor	High initial stiffness, requires good grouting.
Drill steel	v good	poor	high	poor	High shear resistance.
Yielding tendon	fair	v good	med.	fair/good	Good yieldability.
Cable tendon >4m	fair	fair/poor	high	good	Yieldability + flexibility, may require good grouting.
Wire loops <3m	fair	fair/poor	med.	good	
Sets					
Arches + cribbing	poor	fair	high		Areal coverage in poor areas.
Fabric					
Mesh and lace	poor	good	low		Areal coverage + flexibility, labour intensive.
Reinf. s/crete (50 mm)	good	fair/poor	med.		Areal coverage + limited deformability.
Reinf. s/crete (50 mm) & lace	good	good	good		Areal coverage + fair deformability.
Un-reinf. s/crete (50 mm)	good	poor	med.		Areal coverage in areas of low deformation.

Figure 6.2.4 depicts the results of a programme of shear tests carried out on a selection of tendons which were installed in a 40 mm diameter hole drilled in rock and encased in a steel pipe. The initial shear stiffness up to a load of approximately 50 kN is similar for all types except for the Split Sets. Above 50 kN the results are quite variable, with the 16 mm steel rope offering the greatest shear resistance of the types tested. The yielding cone bolt tolerated significantly more shear deformation than the other tendons.

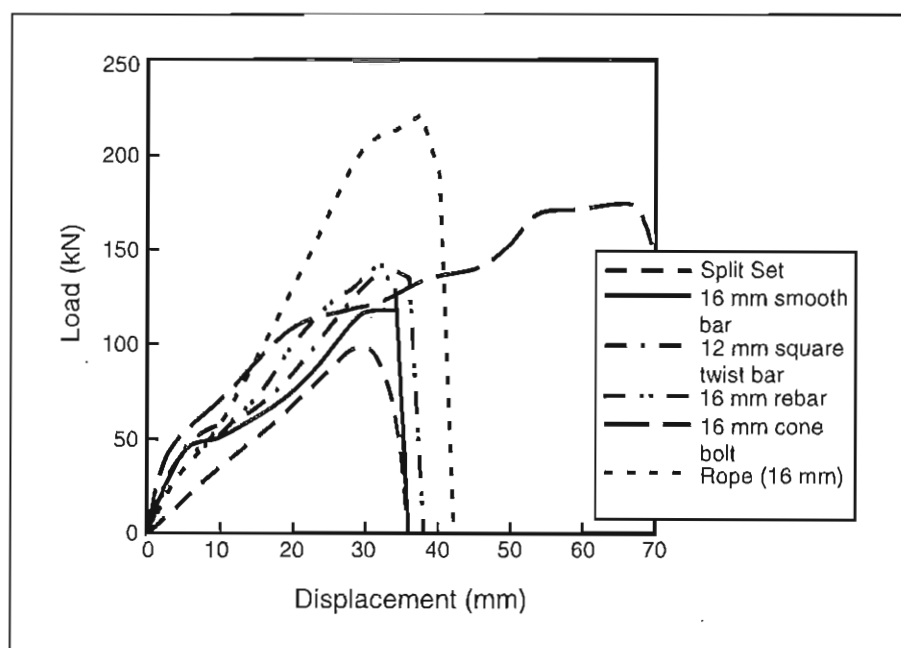


Figure 6.2.4 Behaviour of tendons under shear

6.3 DESIGN CONSIDERATIONS

The design of a tunnel support system must cater for the anticipated mechanism of rock mass instability and the interaction of the elements of the support system with the rock mass. This interaction will be controlled by the rock mass characteristics, and the loading conditions.

6.3.1 Geotechnical Controls

The structure of the rock mass and the conditions of the discontinuities will govern the extent of instability around an excavation (consequently choice of tendon length), and also the interaction of tendons within the rock mass (tendon spacing). The structure of the rock mass is determined by the relative spacing and orientation of the natural discontinuities (jointing and bedding) and any superimposed fracture patterns, which together create the blocky nature of the medium. The condition of a discontinuity is a function of the roughness of the surface and the presence of any alteration, in-filling or water within the discontinuity. Together these influence the effective friction and cohesive properties of the discontinuity.

The more incompetent the rock mass (high frequency of discontinuities, low effective friction) the greater the potential extent of instability and the poorer the interaction between support tendons and the rock mass. Thus, the presence of abnormally poor ground conditions, due for example to the proximity of high-stress mining abutments or the traversing of faults/dykes/joint-swarms, will necessitate a careful evaluation of the design of the support system.

The orientation of the tendon installation relative to the orientation of the disconti-

nities within the rock mass will also influence the extent of interaction between the elements in the support system. In particular, it is found that dominant structures sub-parallel to the axis of tendon installation result in a significantly reduced extent of support interaction. This will result in increased instability between the tendons and a weakening of the overall reinforced rock mass structure unless addressed by reducing the spacing between tendons. Examples of this behaviour include tunnel sidewalls in a highly bedded rock mass where argillaceous layers are associated with the bedding planes. Under these conditions good areal coverage support is an important requisite.

6.3.2 Stress Environment

The stress environment determines whether the stability of an excavation will be controlled mainly by geotechnical structures or by stress-induced fracturing. Case studies in deep South African gold mines in generally competent ground have indicated that the onset of fracturing may be anticipated at an RCF value of about 0.7. This equates to an absolute tangential stress level of approximately 0.7 times the uniaxial compressive strength (UCS) of the intact rock, equivalent to a field stress level of approximately 0.3 times the UCS. This fracturing will be accompanied by dilation of the rock mass, thus increasing the necessity for yield capacity within the support systems.

At RCF values in excess of about 1 in strong brittle rock, '**strain bursting**' can occur on the development face. Consequently the necessity arises for improved areal coverage support in the immediate face area to protect the development crew, or consideration of the use of tunnel preconditioning techniques under these circumstances.

In very **low-stress** environments, such as shallow excavations or excavations developed immediately above or below mined-out areas, the low clamping stresses may result in loosening of the rock mass in the immediate rockwall of the excavation. The use of very stiff or active support elements is therefore required in these environments.

A **change in the stress field** acting on an excavation will also influence its stability. This may result from an increase in the extent of fracturing around an excavation due to a stress increase, or induced shearing and dilation within the rockwall due to a large relative reduction in one of the stress components. Case studies in deep gold mines have indicated resulting dilations of the order of 0.7 mm per metre of unstable sidewall rock mass per MPa increase in vertical stress, or 0.5 mm per metre of unstable hangingwall rock mass per MPa reduction in vertical stress. These deformations, whose magnitude may well differ in different geotechnical areas, will define the required yield capacity of the support system for the excavation over its anticipated life.

6.3.3 Seismicity

The influence of seismicity on excavation support design will be dependent on the mechanism of interaction of the dynamic stresses with the excavation peripheral rock mass. This may vary from seismically induced falls of ground due to the development of tensile stresses within the hangingwall or upper sidewalls of the excavation; or dynamic expulsion of blocks due to the direct incidence of high ground velocities; or the initiation of failure within the peripheral rock mass due to induced dynamic

stresses. In the South African deep level mining environment, the rockburst damage to service excavations is generally due to seismic events occurring on major mining abutments or fault/dyke structures. Large events can cause sporadic damage to tunnels over a widespread area. The damage is often concentrated in areas of unfavourable geotechnical condition where an upgrading of support should have been installed or where the condition of support has deteriorated due mainly to corrosion. Serious anomalous damage can also occur in short portions of a tunnel, where conditions and support designs appear to be completely uniform and of a reasonable standard. This demonstrates current deficiencies in understanding of rockburst mechanisms and the complexities of transmission and reflection of seismic energies through the rock mass.

Design of the support system should consider the extent of interaction of the support elements with the rock mass and their load-deformation capacity, as these will need to take into account the degree of dynamic loading and deformation to be expected as well as the overall rock mass condition.

6.4 DESIGN METHODOLOGIES

The design of support systems for tunnels must consider the rock mass environment in which the excavation is to be sited to ensure the desired level of stability for the required utilisation of the excavation. The analysis of the support requirements will depend on the availability of data and the degree of understanding of the potential rock mass failure mechanisms to ensure adequate interaction of the support elements with the rock mass. As the level of confidence of understanding of the rock mass characteristics decreases, so the design process may shift from a complete structural analysis, to the use of classification systems, to simple empirical design rules. At each stage, greater *a priori* assumptions of the rock mass characteristics need to be made and thus the design process needs to become more conservative and costly, in order to accommodate the uncertainties involved.

6.4.1 Structural / Mechanistic Analyses

Design of a support system based on structural analyses considers the detailed properties of discontinuities and the mechanisms of potential rock mass failure, as well as the physics of the support/rock mass interaction.

(a) Key block analysis. In relatively simple rock mass structures (generally in low stress environments where individual joints can be analysed to determine potentially unstable blocks), a tendon system can be designed based on static analysis of the support capacity in relation to the unstable mass. Generally it is assumed that simple fall out, or sliding, of the block will occur under gravity loading and the tendons are uniformly loaded. However, complex block rotations may result in sequential loading, and failure of the support units under these conditions. A suitable factor of safety thus needs to be used, even in this simplest of design procedures.

(b) Containment of unstable peripheral rock mass. As rock mass complexity increases, the instability of an excavation tends more towards a general unravelling of the rock mass than the fall out of individual key blocks. Where anchorage of the

tendon elements is in excess of the depth of the potential instability, then the support design may be based on uniform confinement of the unstable rock mass volume. The majority of design procedures assume 'tributary area' loading of the tendons, and the results are expressed as the required support resistance (kN/m^2) of the support system. Under dynamic loading conditions it is the energy associated with this tributary area rock mass volume that must be contained by the support system.

However in highly discontinuous rock masses, there is still a tendency for unravelling of the rock mass between the tendon elements. Areal coverage (fabric) support systems need to be designed to minimise this unravelling process and thus maintain the overall integrity of the rock mass. Design of the support system should consider the extent of potential instability between the rock bolt units and thus the demand on, or necessity for, the fabric support. This will depend on the defined rock mass characteristics and the anticipated loading environment (static or dynamic).

The energy absorption requirement E (kJ/m^2) of the support system is given by

$$\text{Hangingwall} \quad E = \frac{1}{2} m v^2 + m g h$$

$$\text{Sidewall} \quad E = \frac{1}{2} m v^2$$

where m is the mass/ m^2 of potentially ejected rock based on the defined unstable rock mass thickness (taking into account the extent of interaction with the rock mass and the rock mass density); v is the anticipated peak ejection velocity; h is the allowable yield in the support element, or yield capacity at a given point in time and g is gravitational acceleration.

(c) Design of a reinforced rock mass structure. In poor ground conditions the depth of instability of the rock mass around a tunnel may be in excess of the practical tendon length. The design consideration under these conditions is to create a rock mass structure (a 'reinforced shell') in the periphery of the excavation. The capacity of this structure to carry load and resist deformation must be suitable for the anticipated loading environment. Of critical importance is the interaction of the tendon elements within the rock mass to increase the inherent strength of the structure. In a highly bedded rock mass where installation of the tendons is perpendicular to the laminations, this may need to include means for the clamping of the individual layers to create a beam of sufficient thickness, based on beam analysis, to withstand the loading likely to be imposed. In complex rock mass structures, the interaction between the tendons to maintain the integrity of the structure becomes increasingly important. The use of high quality, relatively stiff fabric support systems, such as shotcrete, greatly assist in maintaining the stability of the rock mass between the tendon reinforcement under these conditions. The design of support systems for this environment is implicitly catered for in most empirical design methodologies. Careful evaluation of the mechanistic interactions between the rock bolt reinforcements for specific rock mass characteristics needs however to be made. Research in this area is nearing completion: the proposed design methodology analyses the load deformation characteristic, and thus the support resistance and energy absorption, based on an estimation of the support-rock mass system as opposed to the evaluation of support components in isolation. The importance of the rock mass structure on the interaction of rock bolts in the design of the support system has been discussed under section 6.3.1.

6.4.2 Rock Mass Classification Systems

Rock mass classification systems attempt to define the characteristics of the rock mass based on a simplified assessment of critical rock mass parameters. The following components are found in most classification systems (c.f. Chapter 10.3.5):

- the strength of intact rock samples,
- the number and spacing of natural joint sets in the rock,
- the frictional properties of the natural joints,
- the effect of ground water.

The level of data available is generally insufficient to conduct detailed structural analyses of the rock mass and potential failure mechanisms. The instability of the rock mass is generally assumed to be an unravelling mechanism which necessitates the implementation of systematic support systems. The structural interaction between the rock bolt reinforcement and the mechanism of rock mass instability is generally not clearly defined. The rock mass classification may then be used to define appropriate support systems based on case studies within the applicable geotechnical environment. In using classification systems, one must consider the geotechnical environment for which the classification system was established, the level of support technology available, and the required relative safety level of the excavation based on its envisaged utilisation.

Classification systems have been widely utilised in civil engineering tunnelling exercises (RMR, Q systems), and cater well for low stress environments where geological discontinuities control the instabilities involved. In addition, the level of safety of civil constructions is generally required to be far greater than that expected in mining applications. This can lead to the definition of (civil engineering) support requirements that would be considered ultra-conservative for mining applications. Attempts have been made to relate these (civil engineering) systems to mining applications (MRMR) by the empirical evaluation and adjustment of the classifications of the rock mass in relation to typical successful mining support practices.

In deep mining environments, special consideration must be made for large values or changes in the stress field, and the risk of seismicity with potential for rockburst damage. In such environments a design methodology for the control of tunnel condition, and the recommendation of suitable support systems, is based on the classification of the rock mass by the Rockwall Condition Factor (RCF):

$$RCF = (3\sigma_1 - \sigma_3) / F \cdot \sigma_c$$

where σ_1 and σ_3 are the major and minor principal stresses within the plane of the excavation cross section; and F is a factor to represent the downgrading of σ_c (the uniaxial compressive strength) for the representative rock mass condition and excavation size. In good-quality quartzites, F takes on a value of unity. In a highly discontinuous rock mass, a suitable value for F is approximately 0.5, and in large excavations ($> 6 \times 6$ m), F may be further downgraded by 10-20%.

The formulation of the RCF is based on a simple comparison of the maximum induced tangential stress of an assumed circular excavation to the estimated rock mass strength. The empirical relationships between the RCF and recommended support systems is based on extensive field studies of Witwatersrand gold mine tunnels

(3 x 3 m). In general it was found that for $RCF < 0.7$, good conditions prevailed with minimum support requirements (Table 6.4.1); for $0.7 < RCF < 1.4$, average conditions prevailed with moderate support system requirements (Table 6.4.2); and for $RCF > 1.4$, poor ground conditions prevailed with special support requirements (Table 6.4.3).

Empirical relationships have been derived between the RCF and the potentially unstable rock mass thickness for competent rock masses ($F = 1$) due to fracturing. It should be noted that this depth represents the potential unstable block height and will be less than the total depth of fracturing. These guidelines indicate that at $RCF = 0.7$, the anticipated thickness of unstable rock mass to be supported is approximately $0.7 \times$ the radius of the excavation, and at $RCF = 1.4$ this thickness is approximately $1.2 \times$ the radius. Under conditions of seismic loading, the increased extent of instability due to the transient dynamic stresses must be considered in the support design.

[It is of interest to note that the above value ranges of the RCF criterion may not apply in Bushveld Complex mines where the rock mass is igneous as opposed to the brittle quartzites encountered in the gold mines. RCF values of over 1.4 have been calculated for tunnels where conditions were good, with very little stress fracturing observed. Unless experimental error is to blame, the only reasonable explanation for this seems to be that the Bushveld rocks are better able to redistribute high induced stress concentrations than the Witwatersrand quartzites.]

The support recommendations in Tables 6.4.1 – 6.4.3 are intended to be indicative and not necessarily *prescriptive*. Actual support requirements in specific geotechnical environments vary widely, and may justifiably fall short of, or exceed, the values suggested here.

Table 6.4.1 Support recommendations for good ground conditions ($RCF < 0.7$).

Case	Primary support (Typical requirements)	Secondary support (Typical requirements)
Static stress conditions	'Spot' support where necessary (rock bolts, etc.)	'Spot' support where necessary (rock bolts, etc.). Light mesh where ground conditions are inherently poor.
Stress changes anticipated	'Spot' support where necessary (rock bolts, etc.)	Fully grouted tendons > 1.5 m in length, installed on basic 2 m pattern. Support resistance: $30 - 50 \text{ kN/m}^2$. Rope lacing (sidewalls only) plus light mesh where ground conditions are inherently poor.
Seismic activity anticipated	Tendons > 1.2 m in length, installed as close to face as possible, on basic 2 m or 1.5 m pattern. Support resistance: $30 - 50 \text{ kN/m}^2$.	Fully grouted (pref. yield) tendons > 1.5 m in length, installed on basic 2 m pattern. Support resistance: 50 kN/m^2 . Light mesh and lace (hangingwall and sidewalls)

Table 6.4.2 Support recommendation for average ground conditions ($0.7 < \text{RCF} < 1.4$).

Case	Primary support (Typical requirements)	Secondary support (Typical requirements)
Static stress conditions	Rock bolts or tendons > 1.5 m in length, installed as close to face as possible on basic 2 m pattern. Support resistance: 30 - 50 kN/m ² .	Steel strapping or rope lacing integrated with primary support tendons; or shotcrete.
Stress changes anticipated	Fully grouted tendons or steel ropes > 1.8 m in length, installed as close to face as possible, on basic 2 m or 1.5 m pattern. Support resistance: 40 - 60 kN/m ² .	Medium strength rope lacing and moderate strength wire mesh integrated with primary support tendons.
Seismic activity anticipated	Fully grouted (pref. yield) tendons or steel ropes > 1.8 m in length, installed as close to face as possible, on 1.5 m or double 2 m pattern (Fig.6.2.3). Support resistance: 80 - 110 kN/m ² .	Strong rope lacing and strong, stiff wire mesh integrated with primary support tendons, plus optional shotcrete.

Table 6.4.3 Support recommendation for poor ground conditions ($\text{RCF} > 1.4$).

Case	Primary support (Typical requirements)	Secondary support installed by specialist crews close to face (Typical requirements)
Static stress conditions	(If necessary) shotcrete to face, then fully grouted tendons or steel ropes, length > 1.8 m on basic 1.5 m pattern, as close to face as possible. Support resistance: 80 - 110 kN/m ² .	Moderate strength steel wire mesh usually with additional tendons but may be integrated with primary support, optional shotcrete.
Stress changes anticipated	(If necessary) reinforced shotcrete to face, then fully grouted steel ropes or yielding tendons, length > 1.8 m on basic 1 m or double 2 m pattern, as close to face as possible. Support resistance: 120 - 230 kN/m ² .	Medium strength rope lacing and moderate strength wire mesh with additional tendons but may be integrated with primary support. Add integral shotcrete in long life tunnels. If necessary, additional hangingwall support comprising grouted steel ropes.
Seismic activity anticipated	(If necessary) reinforced shotcrete to face, then fully grouted steel ropes or yielding tendons, length > 2.3 m on basic 1 m pattern, as close to face as possible. Support resistance: 220 - 290 kN/m ² .	Strong rope lacing and strong, stiff wire mesh with additional tendons but may be integrated with primary support. Add integral shotcrete in long life tunnels. If necessary, additional hangingwall support comprising grouted steel ropes.