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# Glossary

## Abbreviations

| | |
|---|---|
| 2D | two dimensional |
| 3D | three dimensional |
| BTS | Brazilian Tensile Strength |
| CM | continuous miner |
| CMRR | coal mine roof rating |
| D&B | drill and blast |
| DME | Department of Minerals and Energy |
| FOG | fall of ground |
| GP | grip factor |
| IST | impact split test |
| ISR | impact splitting unit rating |
| PoF | probability of failure |
| PoF | probability of stability |
| RMR | rock mass rating |
| RQD | rock quality designation, usually determined by accumulating all pieces of core greater than 100 mm in a borehole and expressing the value as a percentage of the length of hole or portion of the hole |
| SM | safety margin |
| UCS | uniaxial compressive strength |
| UTS | ultimate tensile strength |

## Symbols and technical terms

| | |
|---|---|
| $\rho$ | the density of rock |
| $\mu$ | coefficient of friction between the layers |
| $\tau_{max}$ | maximum shear stress |

| | |
|---|---|
| $\sigma_1$ | in rock testing, commonly the axial stress |
| $\sigma_1$, $\sigma_2$ and $\sigma_3$ | major, intermediate and minor principal stress |
| $\sigma_{xx}$ | maximum tensile stress |
| $\sigma_3$ | in rock testing, commonly the confining stress |
| $\beta$ | reliability index |
| $\eta_{max}$ | maximum deflection |
| $\tau$ | contact shear strength |
| abutment | the solid area at the edge of a mined out area |
| bord | roadway driven in orebody or seam and specially defined as that area between two pillars, which is not included in the definition of an intersection |
| $B$ | bord width |
| $B_S$ | bond strength |
| core | cylindrical shaped rock retrieved from a borehole |
| $D$ | nominal diameter of the anchor or borehole |
| $d$ | distance between the rows of roof bolts |
| density | mass per unit volume |
| discontinuity | geological or mining induced breaks in the rock mass |
| $E$ | elastic modulus |
| extensometer | measures deformation within the rock mass by means of anchors placed within a borehole |
| extraction ratio | the ratio of mined to unmined ground |
| face | the end of a panel which is advanced during mining |
| floor | the rock mass below the excavation |
| fracturing | discontinuities forming as a result of mining |
| $g$ | gravitational acceleration (9.81 m/sn$^2$) |
| geomechanical testing | test to determine the physical properties of a geological material |
| geotechnical condition | an evaluation of the nature and condition of the geological discontinuities and rock material contained in a rock mass |
| $G(X)$ | performance function |

| | |
|---|---|
| $h$ | mining height |
| $h_1$ | height of roof softening |
| intersection | The area where two roadways meet or cross one another |
| ISRM standards | international standards for rock mechanics tests set by the International Society of Rock Mechanics |
| joint | geological discontinuity |
| k-ratio | the ratio between the horizontal and vertical stress |
| $L$ | span |
| $L_b$ | distance between the bolts |
| $l_b$ | bond length |
| $n$ | number of bolts per square meter |
| $N_{mc}$ | number of Monte Carlo trials |
| panel | span between the barrier pillars |
| panel span | the mined out span between two adjacent lines of barrier pillars or abutments |
| phi ($\phi$) | friction angle |
| point anchor | a roof bolt anchoring system where the anchor is in contact with the strata for a relatively short distance. |
| Poisson's ratio | lateral strain divided by axial strain, lateral strain being the result of an axial stress |
| roadway | an excavation developed in a coal seam, which encompasses both a bord and an intersection |
| roof | the rock mass above the excavation |
| roof bolt | a steel tendon anchored chemically (resin) or mechanically complete with a nut washer and meeting performance specifications |
| $SF_{sus}$ | safety factor in suspension mechanism |
| $SF_{beam}$ | safety factor in beam building mechanism |
| $SF_{slide}$ | roof bolt sliding safety factor |
| $S_B$ | ultimate tensile strength of a bolt |

| | |
|---|---|
| spalling | slabs that develop as a result of stress or time |
| span | the shortest distance between in-panel pillars or faces |
| tensile stress | normal stress tending to lengthen a body along the direction in which it acts |
| $t_{com}$ | competent layer thickness |
| $t_{lam}$ | laminated lower strata thickness |
| $T_R$ | frictional shear resistance of tensioned roof bolts |
| $T_B$ | shear resistance generated by the bolts |
| tensile zone | a tensile stress field that develops above a panel as a result of mining |
| unit weight | the weight per unit volume. |
| $V$ | shear force |
| $V_{max}$ | maximum shear force |
| Young’s modulus (E) | stress divided by the strain resulting from the stress |

# Chapter 1.0

# Introduction

## 1.1 Background

Coal mining contributes to the energy and chemical industries in South Africa. The total annual production of run of mine coal in 2005 was 273 million tons of which approximately 50 per cent was mined by underground methods (Chamber of Mines of South Africa, 2007). Of the underground production approximately 80 per cent is mined by methods which rely on some form of roof support.

Today, roof bolting is, by far, the most common support system used in South African collieries. Because it is more economic than other methods; it saves material and manpower consumption. Most important of all, roof bolting is more effective and efficient because it is an active support method, utilising the rock to support itself by applying internal reinforcing stresses. Furthermore, rock bolting can be satisfactorily used to meet a variety of geological conditions and various support requirements. Roof bolts are available in many forms and the methods to attach them to the rock mass are almost as varied. Full column single resin bolts, full column slow-fast combination resin bolts, resin point anchors and mechanical anchors are the most widely used support systems used in South Africa. Significant advances have been made over the last 20 years in all elements of roof bolting. The design of roof bolt patterns has also been improved. However, studies into the causes of falls of ground show that falls of ground (FOG) have been the major cause of fatalities in South African collieries since 1970.

The distribution of all fatality and injury rates in South African collieries for the period 1984 to 2004 is presented in Figure 1-1. This figure indicates that although there has been a steady reduction in the rates of both fatalities and injuries in collieries until 2001, the rate of fatalities and injuries have increased since 2001. It is also seen in this figure that over many years the rates of fatalities and injuries fluctuated significantly, therefore the fatality and injury rates are not predictable. The peak in the fatality rate in 1993 in this figure was due to a methane explosion in a colliery, which killed 53 miners.

The cause of fatalities in South African collieries for the period 1995 to 2004 is shown in Figure 1-2. This Figure indicates that for this period FOG has been the major cause of fatalities in South African collieries, and the greatest reduction amongst all other causes for fatalities has been achieved in the FOG since 1996.

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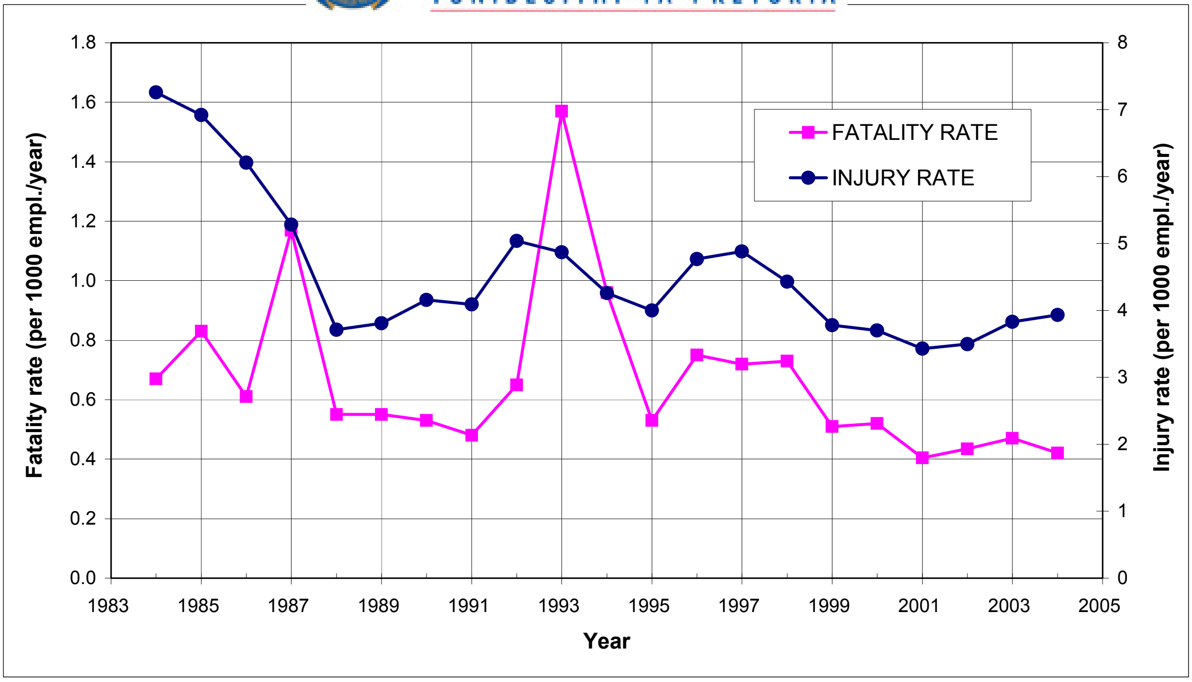


***Figure 1-1 Fatality and injury rates in South African collieries for the period 1984 to 2001***

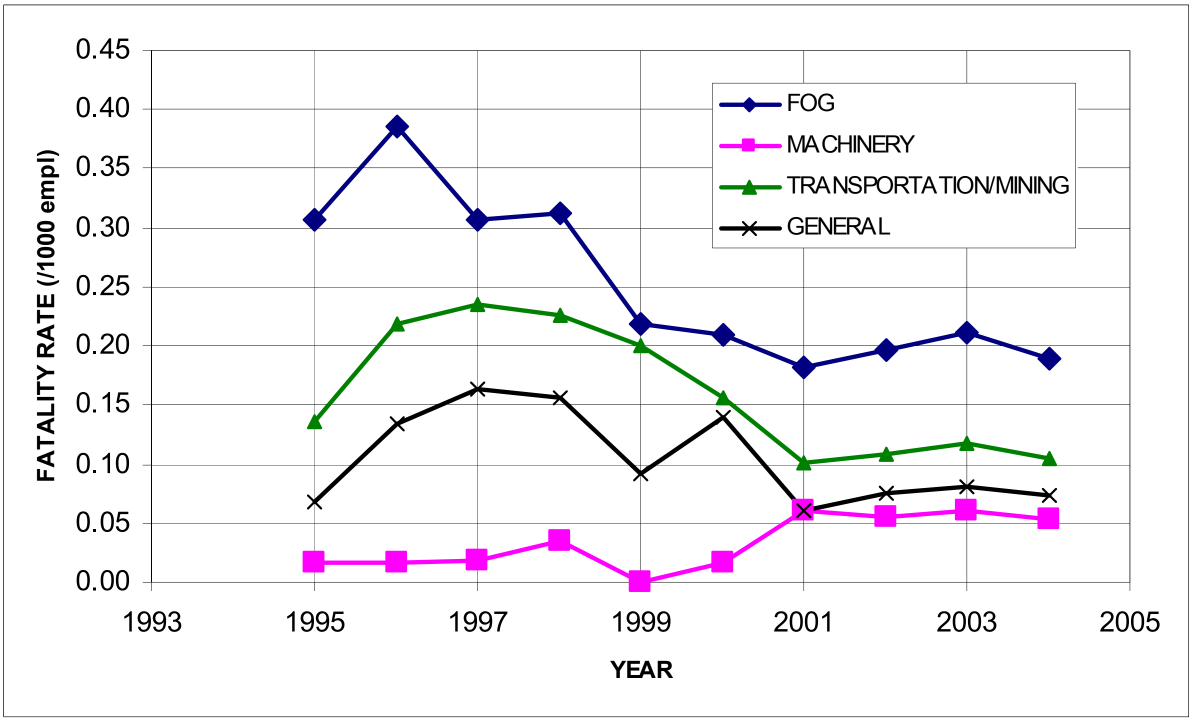


***Figure 1-2 Cause for fatalities in South African collieries for the period 1995 to 2001***

Also, a study conducted by van der Merwe et al. (2001) into the causes of falls of ground in South African collieries showed that the majority of roof falls occur under the supported roof (68 per cent of falls of ground investigated) in South African collieries. These indicate that there is a fundamental problem in the use of correct roof bolting in different geotechnical environments in South African collieries.

An understanding of fundamentals of roof and support behaviour and interaction between them as well as the uncertainties in the elements of a support system will therefore improve the effectiveness of roof bolts installed.

All engineering design incorporates uncertainties in one form or another. In fact, the overall, or total, uncertainty associated with any particular design may incorporate one or more of the following:

- uncertainties due to variabilities of material properties;
- inconsistencies associated with the magnitude and distribution of design loads;
- uncertainties associated with the measurement and conversion of design parameters;
- inaccuracies that arise from the models which are used to predict the performance of the design;
- anomalies that occur as the result of support variabilities;
- gross errors and omissions.

While current support design methodologies, which are mainly based on deterministic constitutive relationships, are unable to account for these uncertainties in any quantifiable manner, the probabilistic design approach, which has gained greater acceptance over the last 20 years is able to incorporate these uncertainties. The value of probabilistic, or stochastic, analyses is that, in accounting for uncertainties and errors, they enable the designer to make estimates regarding the reliability and risk of failure associated with a particular engineering design.

## 1.2 Objectives and scope of research

The objective of the research presented in this thesis is to improve the understanding of the fundamental mechanisms of roof behaviour and the fundamentals of roof bolting in South African collieries to provide guidelines and a risk-based design methodology for their amelioration.

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To meet the main objective of the research, the following scope is set:

- Conduct a detailed literature review on the current knowledge of roof bolting.
- Determine the fundamental roof behaviour through *in situ* monitoring and testing programme.
- Determine the support behaviour and uncertainties associated with support elements through *in situ* testing programme.
- Evaluate currently available design methodologies, especially roof classification systems, to determine the ability of them in predicting the uncertainties in the design process.
- Develop a risk-based design methodology, which will incorporate the uncertainties in the design of support systems.
- Test the developed method against a well defined case.

## 1.3 Outline of the thesis

Following this introduction, a detailed literature review on the subject is presented in Chapter 2. Current knowledge in the fundamentals of support design is summarised.

A detailed underground monitoring programme was carried out in 55 sites covering depths from 32 m to 170 m situated in significantly different geotechnical environments. The effect of unsupported cut-out distance on the roof and support performance was also investigated as part of this study. The results from this monitoring programme are presented in Chapters 3 and 4.

As the Chapters 3 and 4 indicated the variable nature of the roof behaviour, geotechnical classification techniques were evaluated to determine their effectiveness in predicting the variations and uncertainties in the design of roof support systems. The results are presented in Chapter 5.

An investigation into the roof bolting elements that are currently being used in South African collieries was conducted in Chapter 6. All support elements, including the resin, roofbolter, roof

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bolts, drill bits were evaluated in this study. Wet and dry drilling, effects of tensioning, hole annulus and rock type were also investigated.

Based on the knowledge gained throughout this study a new risk-based design methodology has been developed in Chapter 7. The application of this design to a well-defined case is also presented in this Chapter.

Summaries of the conclusions drawn from each Chapter of this thesis are given in Chapter 8.

# Chapter 2.0

# Literature review

## 2.1 Introduction

Roof bolting can be ranked as one of most important technological developments in the field of ground control in the entire history of mining (Mark, 2002). It is an essential component in the design of underground excavations and has been used to provide an overall ground improvement scheme since the middle of the last century. Roof bolting has become the primary support system in the coal mining industry and all underground coal mines in South Africa are mined under supported roofs. Roof bolts dramatically reduce the number of fatalities each year and they were initially hailed as “one of the great social advances of our time” (Mark, 2002).

In the early years, the design of roof bolt systems in South African collieries was based on local experience and the judgement of mining personnel. However, significant advances have been made over the last 20 years in the development of resin anchors, tendon elements and installation hardware. As a result, roof bolting systems have been successfully applied to increasingly extreme roadway conditions as technology has improved and design knowledge has grown.

In the last 20 years, monitoring of roadway behaviour has also been undertaken extensively in coal mining operations. Field monitoring, together with laboratory testing and back analyses through the use of numerical modelling, have provided new insight into rock behaviour and the function and performance requirements of rock reinforcement systems.

This section summarises the most commonly used roof-bolting elements and the design methods that have been developed worldwide.

## 2.2 Types of roof bolts

According to Windsor and Thompson (1997), modern roof support practice may be subdivided into three main techniques:

1. Roof bolting;
2. Cable bolting; and
3. Ground anchoring.

These terms are used to describe the practice of using roof bolts, cable bolts, and ground anchors.

Windsor and Thompson (1997) state that these terms have been in widespread use for many years, and that they describe an important concept, namely the relationship between the reinforcement length and capacity. The reinforcement and length–capacity relationship for the three reinforcement techniques are shown in Figure 2-1. The associated scales of instability are listed below:

| | | |
|---|---|---|
| Surface instability | - | 0-3 m-long elements for roof bolts |
| Near surface instability | - | 3-15 m-long elements or cable bolts |
| Deep seated instability | - | 10-30 m-long elements or ground anchors |

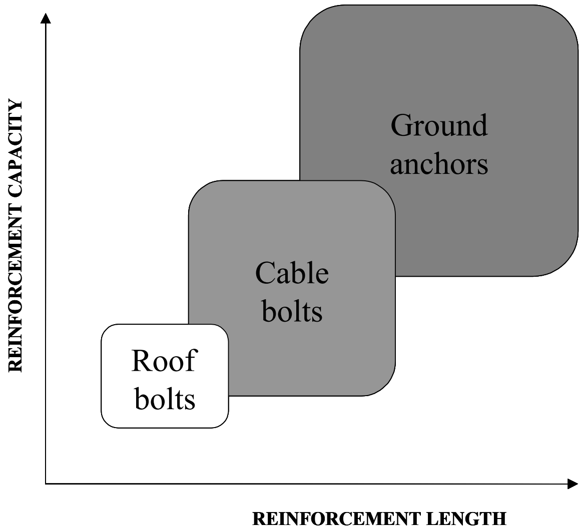


***Figure 2-1 The length-capacity relationships that have evolved for roof bolts, cable bolts, and ground anchors (after Windsor and Thompson, 1997)***

There are eight types of roof bolts used in the South African coal mining industry. These are

1. mechanical anchors;
2. resin point anchors;
3. full-column single-resin-type bolts;
4. full-column slow/fast-resin combination bolts (dual resin system);
5. friction rock stabilisers;
6. wooden dowels;
7. fibreglass dowels; and
8. spin-to-stall resin bolts.

The mechanical anchor bolt, the oldest design in use in underground coal mines, was the main roof support used in the coal mining industry due to the rapid rate of installation. Today, the fully grouted roof bolt is considered superior to the mechanical anchor bolt because of a better anchorage capacity and load transfer capability. Currently, more than 95 per cent of roof bolts installed in South Africa are full-column resin bolts (Minney, van Wijk, Vorster and Koen, 2004). The two main systems are the full-column slow/fast-resin combination, and spin-to-stall systems.

### 2.2.1 Mechanical coupled roof bolts

The mechanical anchor bolt consists of a smooth bar with a threaded anchor end. A mechanical shell anchor attached to the threaded end of the bolt is used to anchor the system. When a torque is applied to the bolt, the force drives a plug against the outer shell, which then expands and sets against the rock in the borehole walls (Figure 2-2). Once the anchor is set, the bolt is then tensioned. Over time, the tension may be reduced as a result of creep or failure of the rock around the anchor. For this reason the mechanical anchor bolt system should be installed in relatively stronger roof rocks.

Van der Merwe and Madden, (2002) state that because of the long free length of the steel tendon, mechanical anchor bolts can stretch when load is applied. It is therefore a soft support, even though it is active by virtue of pre-tensioning. These authors also state that in most coal mine roof types, the anchors start slipping from 30 to 70 kN.

Wagner (1995) stated that, because of high contact stresses which develop at the position of the end anchor, mechanical anchors should be used in rock strata that have a uniaxial compressive strength of more than 50 MPa.

The strength of rock required for mechanically end anchored bolts has also been investigated by Windsor and Thompson (1997). They found that the mechanical performance of the anchor may be estimated using the equilibrium of the forces on the components of the anchor system as shown in Figure 2-3.

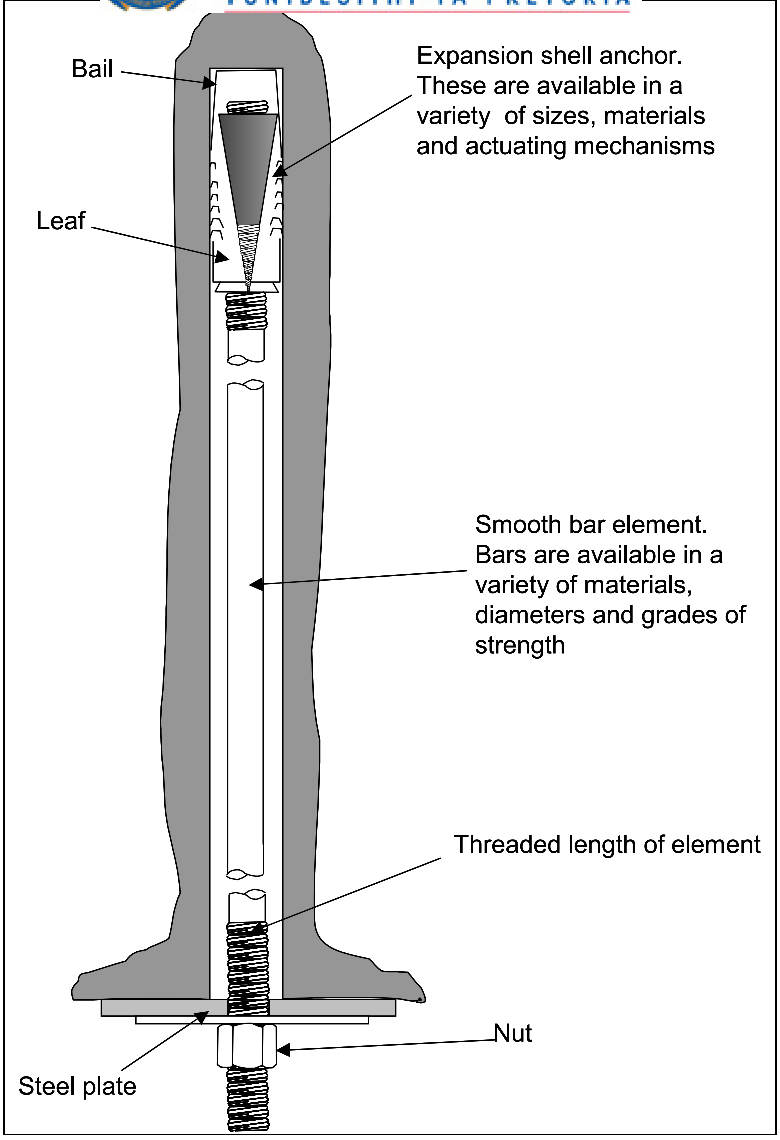


***Figure 2-2 Mechanical anchor bolt***

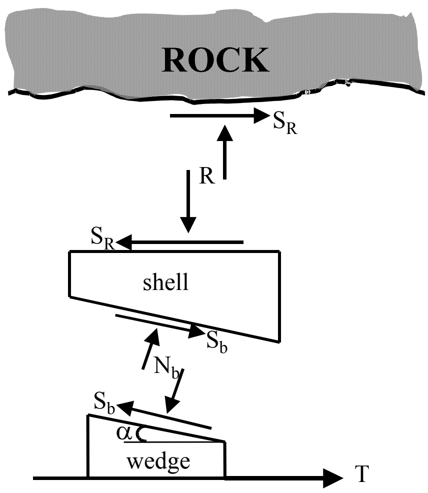


***Figure 2-3 Forces acting on the components of an expansion shell anchor (after Windsor and Thompson, 1997)***

The radial ($R$) and longitudinal shear force ($S_R$) at the interface between the shell and the rock can be converted to approximate equivalent normal $(\sigma_r)$ and longitudinal $(\tau_r)$ stresses with the use of the following equations:

$$\sigma_r = \frac{T}{\pi DL \tan(\alpha + \phi_b)} \qquad [2\text{-}1]$$

$$\tau_r = \frac{T}{\pi DL} \qquad [2\text{-}2]$$

where $D$ is the nominal diameter of the anchor or borehole
$L$ is the length of the shell in contact with the rock
$T$ is tension on the bolt
$\phi_b$ is the contact friction angle (degree)

The radial stress predicted by Equation [2-1] assumes the force is distributed equally around the circumference of the borehole for the total length of the leaves. In reality, the stresses will be greater than this estimate as a result of a non-uniform distribution of the stresses. Also, in hard rock, the teeth in the leaves will initially be in contact with the rock, and the contact stresses will be much greater and bring about local failure. At higher axial forces, the average radial stress will be given approximately by Equation [2-2].

The suitability of an expansion shell anchor for a particular rock type can be assessed with the use of Equations [2-1] and [2-2]. For example, these equations can be used to calculate the maximum radial and longitudinal stresses based on the strength of the tendon. The radial stress may be used to estimate the stresses induced in the rock near the borehole wall and these can, in turn, be compared with the compressive strength of the rock. Shear stresses induced at the borehole wall must also be less than the shear strength of the rock.

Various types of expansions shells are shown in Figure 2-4.

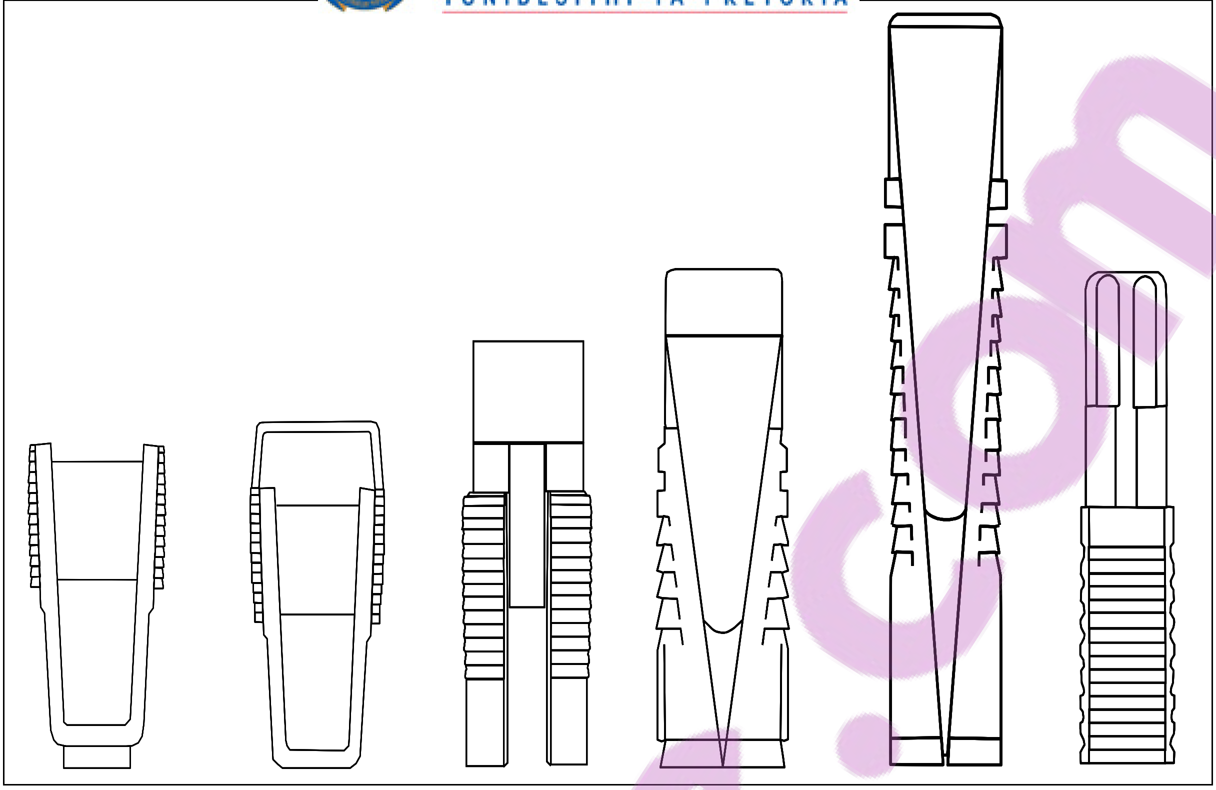

***Figure 2-4 Various expansion shell mechanisms (after Windsor and Thompson, 1997)***

### 2.2.2 Resin point anchors

Resin anchoring of roof bolts with the use of capsules was developed in France during the 1960s (Raffoux, 1971). In principle, the same remarks apply here as for mechanical anchors. The only difference between mechanical anchors and point resin anchors is that the expansion shell is replaced by a fast setting resin (Figure 2-5). This indicates that in areas where the rock is not strong enough to enable mechanical anchors to be installed, point resin anchors may be used.

Resin anchors require more time and care to install than mechanical anchors. Van der Merwe and Madden (2002) described the advantages and disadvantages of the resin point anchor system as follows:

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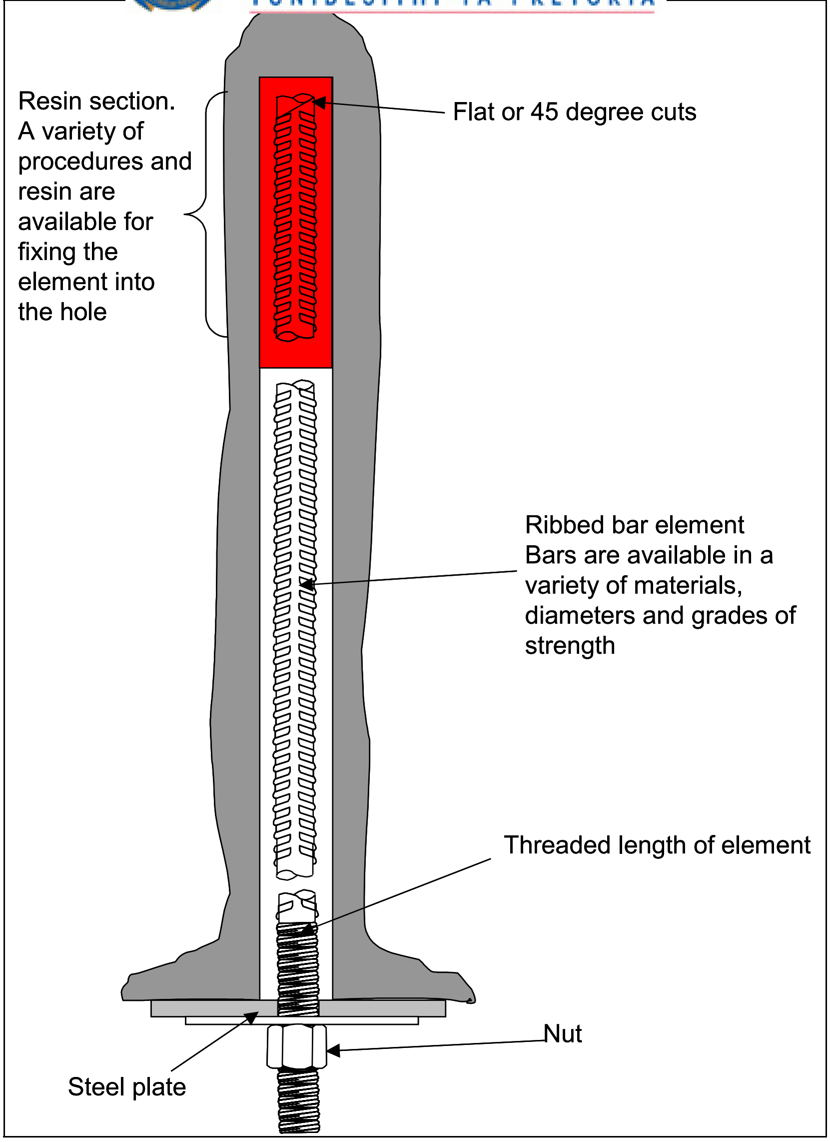


***Figure 2-5 Point resin anchor***

Advantages:

- The anchor resistance can be increased by making the anchorage length longer; and
- The changeover to full-column resin support, should it be required by changing conditions, is less traumatic because operators will already be trained in resin installation.

Disadvantage:

- Point resin anchors cannot be used in friable or burnt coal ribsides, because of difficulties in proper mixing of the resin.

### 2.2.3 Full-column single-resin-type bolts

These are full-column resin bolts of a ribbed bar, anchored with a full-length column of resin obtained from a cartridge (Figure 2-6). This system is considered to be non-tensioned. However, the plate is loaded with stress due to thrust (Karabin and Debevec, 1976). This load can also be increased using the "thrust bolting technique" (Tadolini and Dolinar, 1991), which can apply upwards of 44 kN of initial plate load (Tadolini and Dolinar, 1991). These loads are similar to what is measured in the typical Australian "non-tensioned" roof bolt (Frith and Thomas, 1998).

Because the steel is friction bound to the rock over its entire length, full-column installations allow very little displacement to take place once they are installed, making the system one of stiff support. Furthermore, because the full length of the hole is filled, this system restricts lateral movement between different layers.

Van der Merwe and Madden (2002) described the advantages and disadvantages of the full-column resin system as follows:

Advantages:

- Full-column resin support can be used virtually anywhere;
- It is ideal for any long-term requirement like main developments, underground workshops, etc.;
- Full-column resin support is essential in beam-building mechanisms; and
- It is ideal for the support of laminated roofs.

Disadvantages:

- The support is relatively expensive;
- It requires care to install as operators have to be well trained; and
- Full-column resin anchors cannot be used in friable or burnt coal ribsides, because of difficulties in proper mixing of the resin.

Van der Merwe and Madden (2002) also state that the passive nature of full-column resin can be overcome if bolts are installed close to the face before layer separation occurs.

Mark (2000) found that the total load generated within the resin is generally less than the strength of the steel for bonded lengths of less than 0.61. It was also noted that the bond strength depends on rock strength and other installation parameters.

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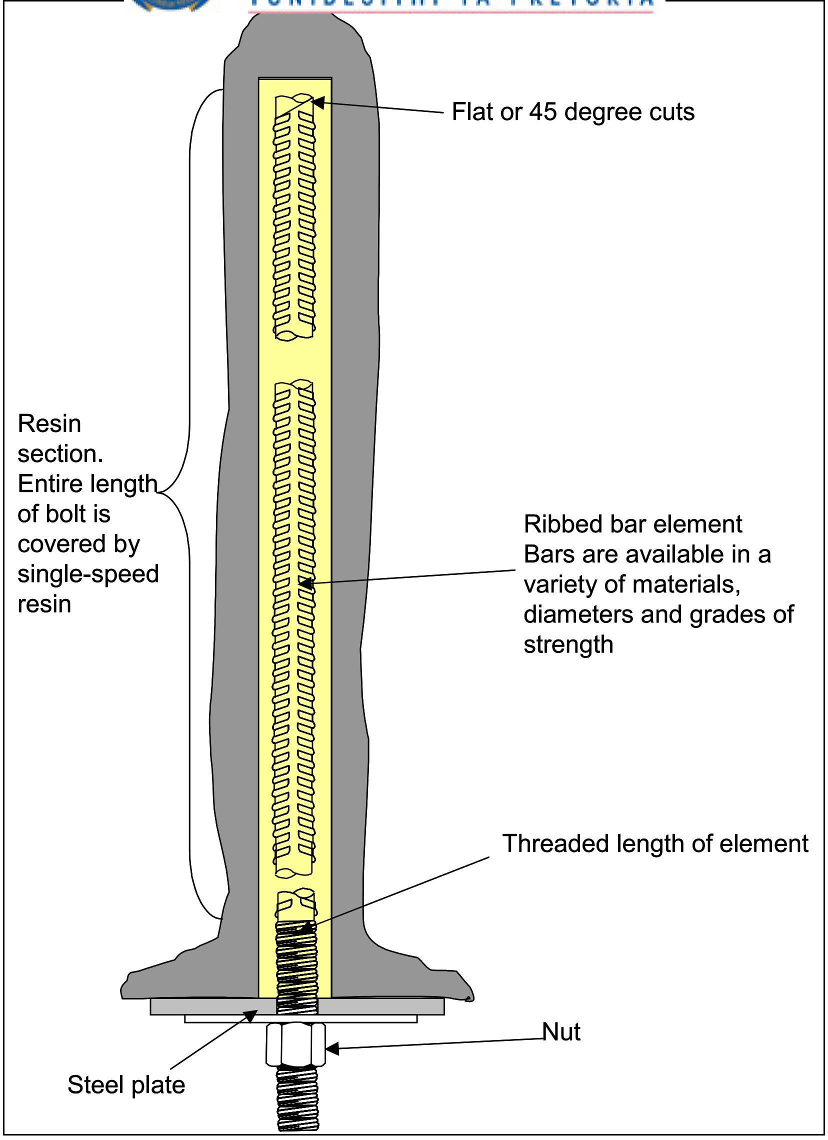


***Figure 2-6 Full column resin bolt***

The stiffness of a full-column single-resin bolt is determined by the load-transfer mechanisms between the rock, the resin, and the bolt (Mark, 2000). Good load transfer exists when very high loads develop in the bolt in response to small ground movements, and these loads are rapidly dissipated away from the zone of roof movement. Poor load transfer can result in:

- Large plate loads;
- Large roof movements before maximum bolt response; and
- Low ultimate bolt capacity, particularly if roof movements occur near the top of the bolt (Fabjanczyk and Tarrant, 1992).

### 2.2.4 Full-column slow/fast-resin combination bolts

This system is the most widely used roof bolting system in South African collieries. It is a stiff and active system (Figure 2-7).

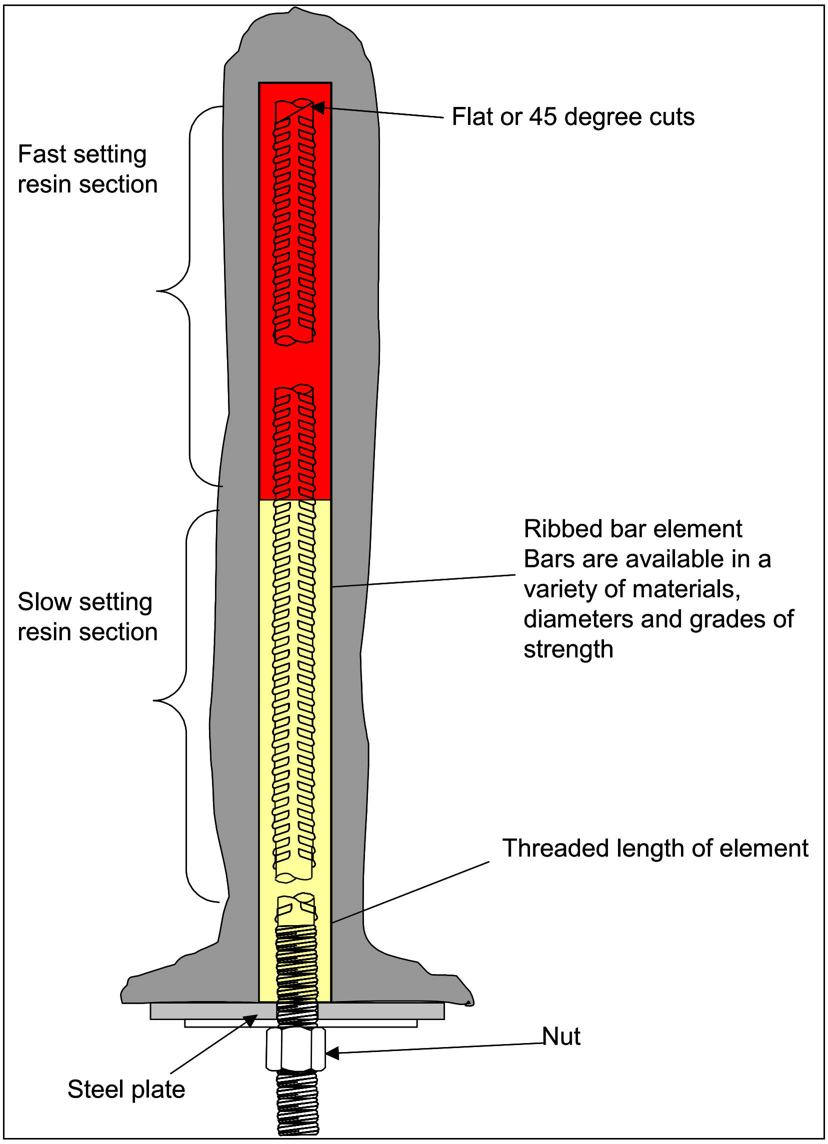


***Figure 2-7 Full-column slow/fast-resin combination bolts (the dual resin system)***

Van der Merwe (1989) found that, in general, slower setting resins tend to result in higher shear strength of the resin/rock contact plane compare to fast setting resin. Also, the slower the resin, the wider the tolerance of the mixing and waiting times.

In full-column installations, it is difficult to install longer bolts (> 1.5 m) with fast resin only. The time taken to push the steel tendon through the resin column (which often has to be done during spinning in order to achieve penetration) sometimes means that the resin at the bottom of the hole will be spun for the incorrect length of time. With very fast setting resins, it was frequently

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found that the resin at the bottom of the hole starts to set before the steel tendon is fully inserted (van der Merwe and Madden, 2002).

On the other hand using only slower resins means that more time is required to complete the installations, which may lead to a loss in production. Van der Merwe (1989) suggests that an appropriate balance needs to be found between the efficiency of the system and the time taken to carry out the installation. For this reason, van der Merwe (1989) suggests the use of dual systems: a single fast capsule is placed at the top of the column, while the remainder of the column is made up of slow resin capsules.

### 2.2.5 Friction rock stabilisers

Friction rock stabilisers are generally passive bolts because they cannot be tensioned. The only friction rock stabiliser realistically available at present on South African coal mines is the Split Set (Buddery, 1989) used for ribside support.

A Split Set is installed by being forced into an undersized hole (Figure 2-8), giving rise to radial forces and, dependent upon the operator and the thrust of the installation machine, a degree of axial load. Strata movement causes frictional forces to be induced along the tendon/rock interface.

Because of the large exposed surface area Split Sets are highly susceptible to corrosion. Most of the corrosion is on the inner surface, and the increased likelihood of tensile or shear failure outweighs any increase in frictional resistance along the bolt/rock interface. For this reason Split Sets should be viewed as temporary support only, unless they are installed in a non-corrosive environment (Buddery, 1989), or post-grouted.

Split Sets are quick and easy to install, but are expensive. In Split Set application, control over hole diameters is crucial. Split Sets are an ideal support for burnt coal and in other applications, for example moulding wire mesh to hollows in roofs and ribsides prior to shotcreting (van der Merwe and Madden, 2002).

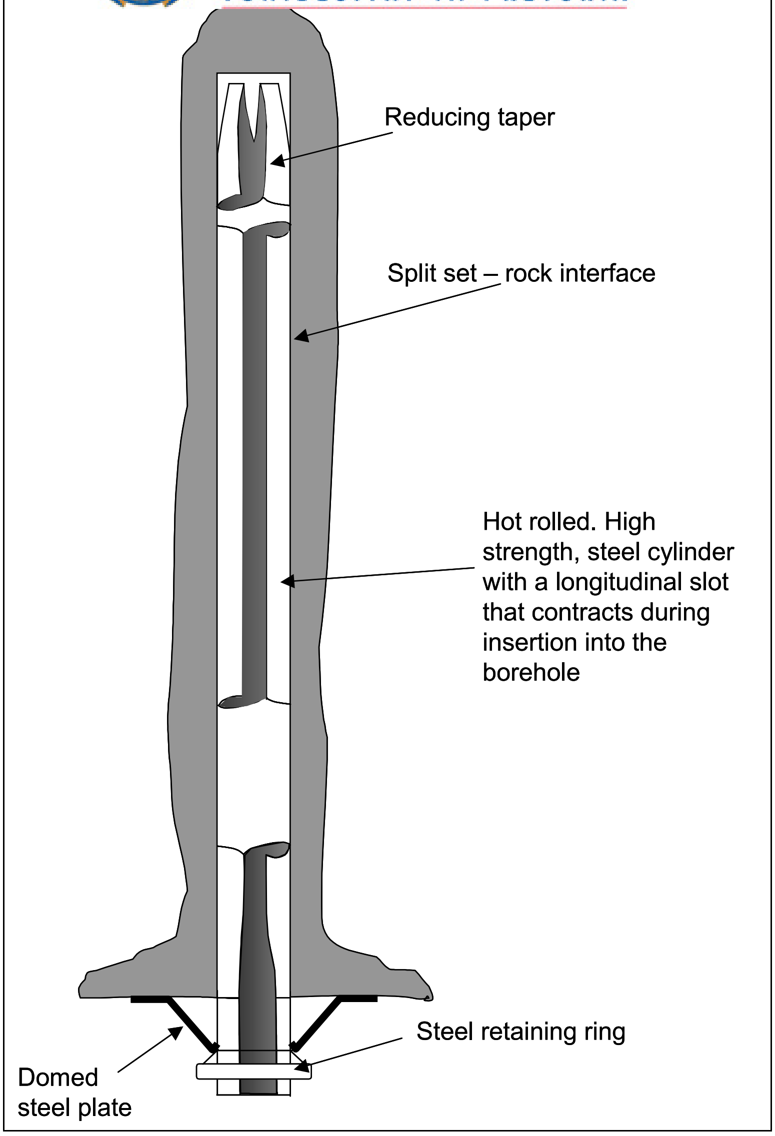


***Figure 2-8 Split Set***

### 2.2.6 Wooden dowels and fibreglass dowels

Dowels are ideal when they are in contact with the host rock along the entire length of the dowel. They are often used as ribside support where steel is not suitable, for example in longwalls, or where stooping is contemplated. Resistance to movement is the result of an "interface fit" provided by either a resin or cement grout filling the void between hole wall and bolt. The grout adheres firmly to the bolt but adhesion to the host rock is not significant. Cement is rarely used in South African collieries (Buddery, 1989).

Dowels are referred to as "passive supports" since they require strata movement before they offer effective support. Tension in dowels is the result of ground movement, which means that frequent manual re-tensioning is unnecessary. Dowels are far less susceptible to corrosion than most roof bolts.

Since a dowel is a non-pre-tensioned device, no purpose is served by a washer unless it is to secure mesh, straps, tapes, etc.

Dowels are very effective in preventing longwall face deterioration in cases where the face is not mined for extended periods (van der Merwe and Madden, 2002).

### 2.2.7 Spin-to-stall system

In the UK and Germany roof bolting was introduced widely in coal mines in the 1980s (Siddall, 1992). The success of this introduction, following earlier failures, depended on the adoption of the high bond strength system, which had been developed in Australia. Because mining conditions in the deeper European mines were even more demanding, further developments to improve the capacity and bonding properties were also made. In these conditions, the importance of ensuring that every bolt is installed correctly led to the development of improved standards and systems for quality control (O'Connor et al., 2002).

Consistent high-quality installation and improved bond strength were also recognised in South Africa, and led to the development of new systems that are unique to South Africa. These are the "reverse-spin" system implemented by SASOL Coal (Postma, 2005) and the "spin-to-stall" system developed by Anglo Coal (Minney and Munsamy, 1998). In the spin-to-stall system, the bolt is spun to mix the resin and spinning continues until the gelling resin increases the resistance, resulting in breakout of a torque nut. The nut runs up the thread and is tightened against the bolt to be installed in approximately 10 seconds. The length of exposed thread provides an indication of the standard of installation.

Although the spin-to-stall system gives a simpler underground operation, it is more demanding on the roof bolting system components. The resin must provide sufficient time for mixing and roof bolt insertion, then transform very rapidly from a fluid to a set state, and develop high bond strength. The properties of the resin, the properties of the roof bolt, the breakout torque of the nut and other parameters are important for developing and optimising the system (O'Connor et al., 2002).

The installation procedure for the spin-to-stall system is shown in Figure 2-9. As can be seen from this figure, there is no holding time in the spin-to-stall system.

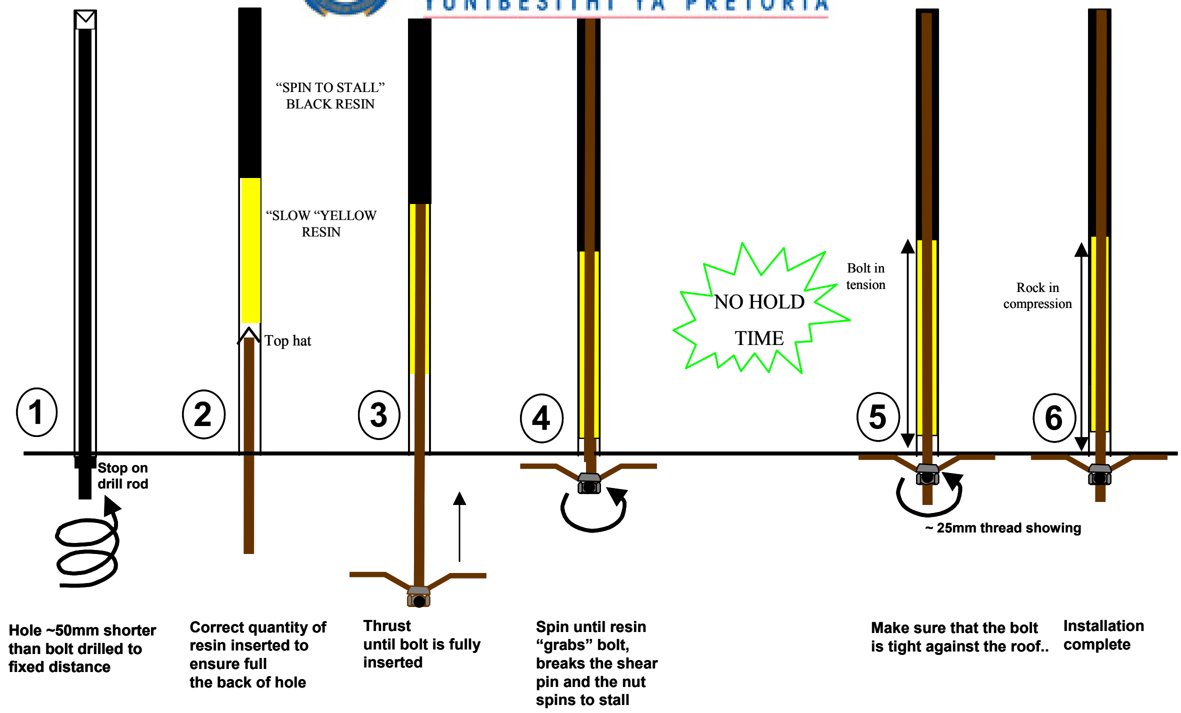


***Figure 2-9 Spin-to-stall installation procedure (after Minney and Munsamy, 1998)***

Minney and Munsamy (1998) reported that the final tightening of the nut may damage the bonding between the bolt and the resin. Therefore, forged-head bolts and shear-pin bolts were recommended in spin-to-stall systems.

Van der Merwe and Madden (2002) state that this type of application may require a denser support spacing to compensate for the weak bond due to the installation procedure. In addition, they state that the spin-to-stall system application should be approached with great caution and should be implemented only after a comprehensive test programme has been carried out.

### 2.2.8 Current guidelines for the selection of roof bolt type

The choice of bolt type depends primarily on the geological condition, the roof rock, and the mining method.

While mechanical anchor bolts are not effective in weak rock, Split Sets are not recommended in corrosive environments. The fully grouted bolts can meet a wider range of roof conditions and support requirements (Smith, 1993; van der Merwe and Madden, 2002). Anchorage is distributed over the grouted length, the resin protects against corrosion and, even if the rock weathers away from the bearing plate, the resin/rebar will continue to hold the rock together. For long-term support, the resin/rebar bolt will always be a better choice (Parker, 2001).

Yassien (2003) made recommendations on the selection of bolt type. Mechanical bolts are recommended for:

- Hard and strong rock as they can resist bit biting and keep the anchorage force;
- Temporary reinforcement systems;
- Conditions where bolt tension can be checked regularly;
- Rock that will not undergo high shear force; and
- Areas away from blast sites where bolt tension may be lost.

Fully grouted bolts are recommended by Yassien (2003) for:

- Areas and conditions where mechanical bolts are not recommended;
- Rock without wide fractures or voids that will cause grout loss; and
- Long-term support of thinly bedded roof strata.

Maleki (1992) proposed the preliminary criterion for selecting bolt types depending on the stress level and rock mass strength by the following formula (Figure 2-10):

$$Rock\ Mass\ Strength = \frac{Uniaxial\ compressive\ strength}{K} \qquad [2\text{-}3]$$

where $K$ equals 1 for massive strata; $K$ equals 2 for cohesive, medium bedded strata; and $K$ equals 3 for thinly laminated, non-cohesive strata.

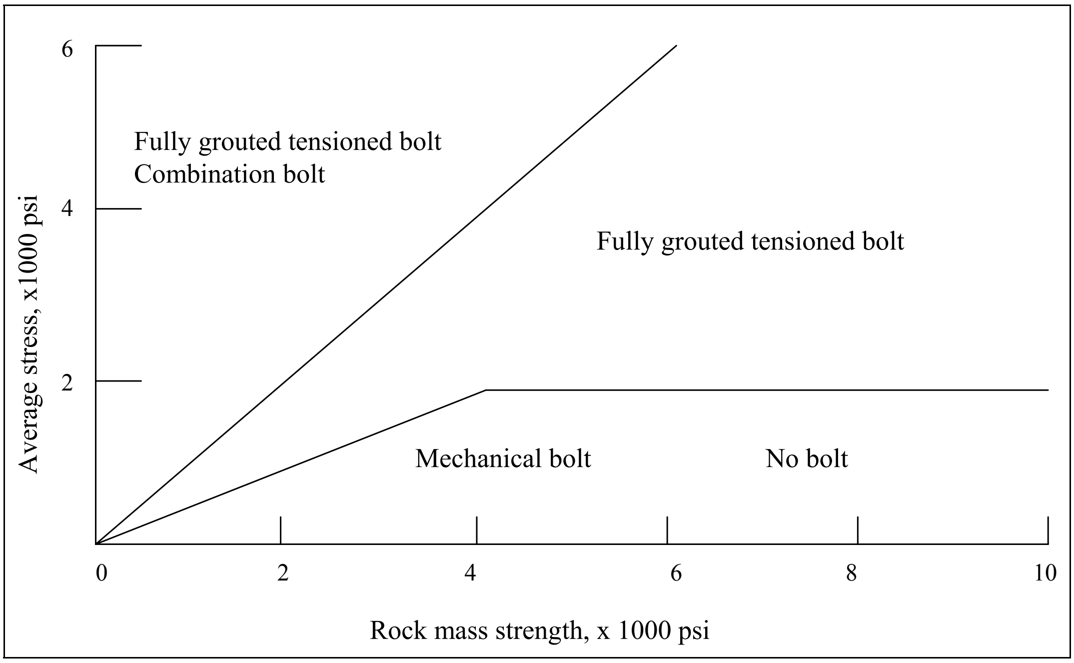


***Figure 2-10 Selection of bolt type (after Maleki, 1992)***

Van der Merwe and Madden (2002) summarised the characteristics of the different support systems that indicate their main areas of applicability (Table 2-1). Table 2-2 lists some of the more commonly encountered ground conditions, and indicates which support systems are best suited to these.

***Table 2-1 Support system characteristics summary (after van der Merwe and Madden, 2002)***

| System | Active/ Passive | Stiff/ Soft | Corrosion resistance | Ease of installation | Pull-out resistance | Where to use |
|---|---|---|---|---|---|---|
| Mechanical Anchors | Active | Soft | Medium | Good | Medium | Short term, unlaminated roof, medium, light load |
| Resin point anchor | Active | Soft | Medium | Medium, requires training | Very good | Short term, unlaminated roof, medium, heavy load |
| Full-column resin (single-resin type) | Passive | Stiff | Good | Medium, requires training | Very good | Long term, laminated roof, heavy load, thick weak roof, close to face |
| Full-column resin- (slow/fast combination) | Active | Stiff | Good | Medium, requires training | Very good | Long term, laminated roof heavy load, beam building, thick weak roof |
| Friction rock stabilisers (Split Set in SA collieries) | Passive | Stiffish | Poor | Good | Poor | Burnt coal ribsides, wire mesh fill-in, thin laminated layers, short term, light load |
| Wooden dowels | Passive | Stiff but weak | Excellent | Easy | Poor | Longwall faces, ribsides in stooping |
| Fibreglass dowels | Passive | Stiff | Excellent | Easy | Good | Burnt coal, joint areas, friable roof, long term, densely populated areas |

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***Table 2-2 Support system suitability (after van der Merwe and Madden, 2002)***

| Roof description | Suitability rating | | |
|---|---|---|---|
| | **Good** | **Medium** | **Poor** |
| Sandstone, occasional false roof | Mechanical anchors<br>Resin point anchor | Split Set | Full-column resin bolts (cost) |
| Sandstone underlain by thin layer of laminated material | Short full-column resin bolts | Resin point anchor<br>Split Set (short term) | Mechanical anchor |
| Thick layer of laminated material | Full-column resin bolts (slow/fast combination)<br>Angled bolts | Resin point anchor<br>Full-column resin bolts (single resin type) | Split Set<br>Mechanical anchor |
| Thick layer of weak material | Full column resin bolts (slow/fast combination)<br>Angled bolts<br>Roof trusses | Full-column resin bolts (single resin type) | Resin point anchor<br>Mechanical anchor<br>Split Sets |
| High horizontal stress | Full-column resin<br>W-straps<br>Long anchors | Resin point anchor | Mechanical anchor |
| Burnt coal, ribsides | Split Set<br>Wire mesh and Shotcrete | Dowels | Any resin anchor<br>Mechanical anchor |

Smith (1993) also investigated the selection of appropriate support for different geotechnical environments and concluded that the selection of bolt type mainly depends on the geological and tectonic conditions and the required lifetime of the bolting system. Smith (1993) established the following guidelines for selecting the appropriate support system for different environments:

1. Mechanical bolts are used in:

   - Harder rock conditions where the rock properties will not adversely affect the gripping force of the anchor;
   - Temporary reinforcement systems;
   - Conditions where bolt tension can be checked regularly;
   - Rock that will not experience high shear forces;
   - Rock that is not highly fractured; and
   - Areas away from blast sites where bolt tension may be lost.

2. Resin bolts are generally used in:

   - The conditions as set out above but where mechanical bolts are not recommended;
   - Permanent reinforcement systems;
   - Boreholes without continuous water run-off problems or with continuous water run-off that would not interfere with installation; and
   - Rock without wide fractures and voids in which significant amounts of grout could be lost.

3. Non-tensioned bolts are recommended in rock that is highly fractured and deformable, as long as adequate bolt installation is feasible. Generally, bolts in more competent strata often require a shorter grout column than do bolts in less competent strata.

4. Tensioned grouted bolts are recommended for use where additional frictional forces, in combination with a grouted column, may enhance roof stability.

Table 2-3 shows the bolt types commonly used in coal mines, non-coal mines, and surface mines in the U.S.A. (Peng, 1984).

## 2.3 Theories of roof bolting support

The main function of roof bolting is to bind stratified or broken rock layers together to prevent roof falls. In order to achieve this objective four basic theories have been established for roof bolting (Wagner, 1985; Buddery, 1989; Peng, 1986; Van der Merwe and Madden, 2002; Mark, 2000).

The four theories are:

- Simple skin support;
- Suspension of a thin roof layer from a massive bed;
- Beam building of laminated strata; and
- Keying of highly fractured and blocky rock mass.

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***Table 2-3 Bolt types commonly used in the U.S.A mines (after Peng, 1984)***

| Types of bolt | Types of anchor | Suitable strata type | Comments |
|---|---|---|---|
| Point-anchored bolts (tensioned) | Slot-and-wedge | Hard rock | Used in the early stages |
| | Expansion shell | | Most commonly used in the U.S.A. |
| | Standard anchor | Medium-strength rock | |
| | Bail anchor | Soft rock | |
| | Explosive set | Lower-strength rock | Limited use |
| | Resin grout | All strata especially for weak rock | Increasing usage recently |
| | Pure point anchor | | Resin length 24 in. |
| | Combination system | | Resin length 24 in. |
| | Combination anchor (expansion shell and no mix resin) | Most strata | Good anchorage with "no mix resin" |
| Full-length-grouted bolt (untensioned) | Cement | Most strata | Disadvantages: |
| | Perfo | | 1. Shrinkage of cement |
| | Cartridge | | 2. Longer setting time |
| | Resin | All strata | Increased use recently especially for weak strata |
| | Injection | | |
| | Cartridge | | |
| Roof truss | Expansion shell | Adverse roof | Recommended for use at intersections and/or heavy pressure area |
| Cable sling | Cement anchor and full-length fraction | Weak strata | Substitute for timber, steel or truss support |
| Yieldable bolt | Expansion shell | Medium-strength rock | It is an expansion-shell bolt with yielding device |
| Pumpable bolt | Resin | Weak strata | Complex installation |
| Helical bolt | Expansion shell | Most strata | |
| Split set | Full-length fraction | Weak strata | Cheap but need special installation equipment |
| Swellex bolt | Full-length holding | Water-bearing strata | Using high-pressure water to swell the steel tube |

### 2.3.1 Simple skin support

A strong, massive roof subjected to low stress levels can be essentially “self-supporting”, meaning that a major roof collapse is unlikely to occur. However, cracks, joints, cross-bedding, or slickensides can create occasional hazardous loose rock at the skin of the excavation (Figure 2-11). Pattern bolting is therefore required to prevent local loose rock from falling, but the bolts may be relatively short and light. Skin control is also an important secondary function of roof bolts, along with the other three support mechanisms (Mark, 2000).

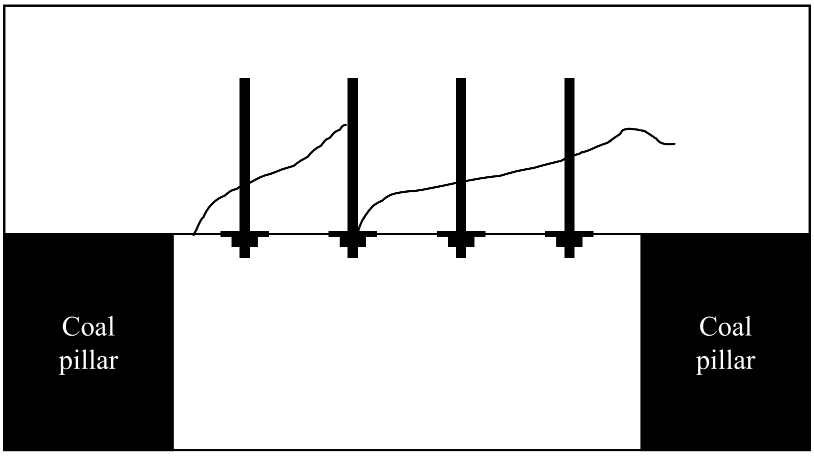


***Figure 2-11 Simple skin support***

### 2.3.2 Suspension mechanism

The suspension mechanism (Figure 2-12) is the most easily understood roof bolting mechanism.

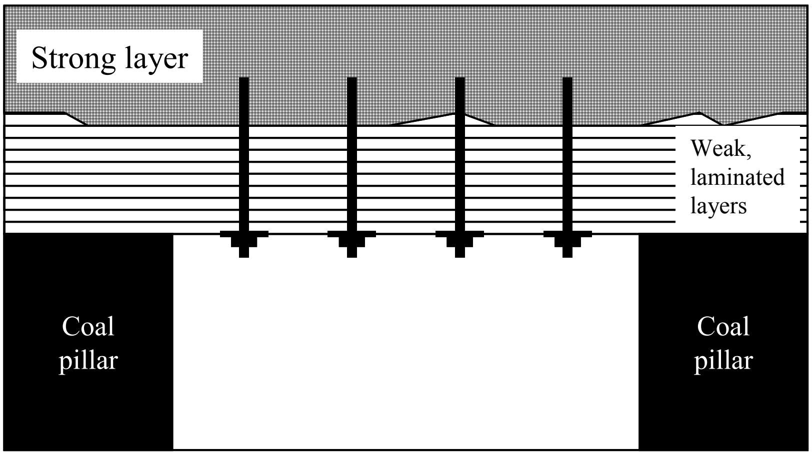


***Figure 2-12 Suspension mechanism***

When an underground opening is made in an environment represented in Figure 2-12, the laminated immediate roof tends to sag and separates from the overlying strong layer. The sag and separation of the immediate roof can be reduced by clamping the laminations together and suspending them from the self-supporting main roof.

Mechanical or resin point-anchored bolts are well suited to this kind of application. With resin bolts, the longer the encapsulation length, the stronger the anchor. The required strength of the anchor depends on the spacing of the bolts and the thickness of the laminated layer. This indicates that the thicker the laminated layer and greater the spacing, the longer the bolts must be (van der Merwe and Madden, 2002).

Wagner (1985) investigated the effectiveness and the applicability of the suspension mechanism in coal mine roofs. It was found that:

- In the case of thin roof beds, the spacing between bolts is critical, with the general rule being that it should not exceed a value of 10 times the thickness of the layer;
- In the case of thicker roof slabs and grouted roof bolts, the length of bolt that is anchored into the competent bed is critical for ensuring sufficient anchorage; and
- In the case of mechanically end-anchored roof bolts, the contact strength of the roof at the position of the end anchor is critical. Contact stresses of 20 to 30 MPa are not uncommon. Such high stresses can only be supported by competent sandstone formations. This fact has to be taken into account in the design of the support system.

### 2.3.3 Beam-building mechanism

In many practical situations, the strata overlying a roadway is thinly laminated. Often there is no competent bed within a distance of a few metres into the roof that could serve to suspend the thin layers on roof bolts. In these cases, the beam-building mechanism, as shown in Figure 2-13, is more effective. As a result, the horizontal movements between these layers will be greatly reduced and the combined thick beam will be more stable (Peng, 1998). Full-column resin bolts are required for this mechanism (van der Merwe, 1998).

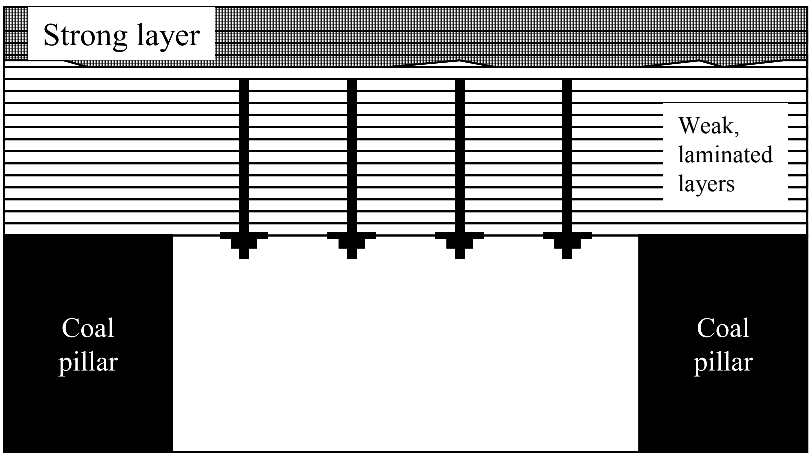


*Figure 2-13 Beam-building mechanism*

### 2.3.4 Keying

When the roof strata are highly fractured and blocky, or the immediate roof contains one or several sets of joints with different orientations, roof bolting can significantly increase frictional forces along fractures, cracks, and weak planes. Sliding and/or separation along discontinuities is thus prevented or reduced, as shown in Figure 2-14. This keying effect mainly depends on active bolt tension or, under favourable circumstances, passive tension due to rock mass movement. It has been shown that bolt tension produces stresses in the stratified roof, which are compressive both in the direction of the bolt and orthogonal to the bolt. Superposition of the compressive area around each bolt forms a continuous compressive zone in which tensile stresses are reduced and the shear strengths of discontinuities are improved, as shown in Figure 2-15 (Luo et al., 1998).

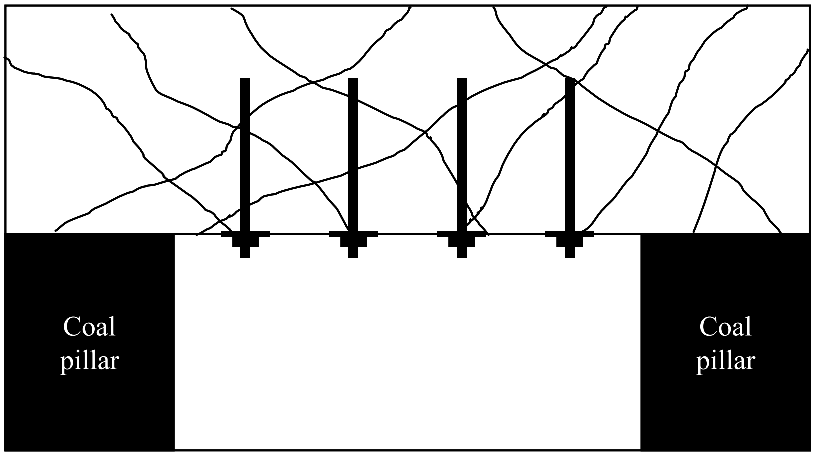


***Figure 2-14 Keying effect of bolting***

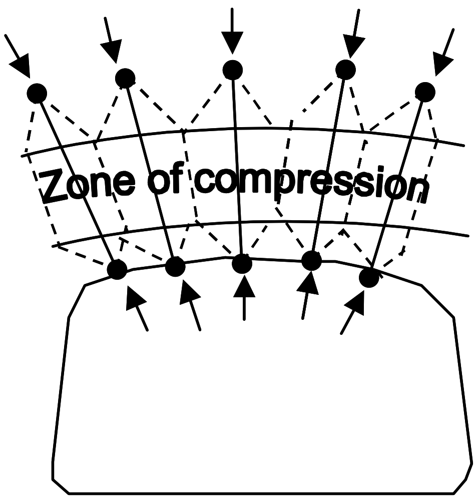


***Figure 2-15 Compression zone created by keying (after Luo et al., 1998)***

## 2.4 Roof bolting design

As in the design of other support systems, the design of a roof bolting system depends on: the nature of the discontinuities and the intact rock; the magnitude and distribution of the stresses induced; support requirements such as acceptable deformation and lifetime of the opening; and the size and shape of the openings. For a complete and appropriate roof bolting system design, the following parameters must be properly determined (Luo et al., 1998):

- Bolt type;
- Bolt length;
- Pattern and spacing of bolts;
- Bolt diameter and anchor capacity;
- Whether pre-tension should be applied or not. If pre-tensioned, the magnitude of the tension should be determined.

In order to achieve the best support system design, the mechanical behaviour of rock masses reinforced by full grouted bolts, i.e. the rock-bolt interaction, needs to be fully understood. The design methodologies for roof bolts can be classified into the following four categories:

- Analytical methods;
- Field testing;
- Numerical modelling;
- Geotechnical classification; and
- Physical modelling.

### 2.4.1 Analytical methods

The oldest, simplest, and probably still the most widely used method for bolt design is dead-weight suspension (Obert and Duvall, 1967; Stillborg, 1986).

A modified version of this design principle (Wagner, 1985) is still being used in South African collieries in the design of suspension methods. The design of roof bolt systems, based on the dead-weight principle, has to satisfy the following requirements:

- The strength of the roof bolt system, $SB$, has to be greater than the weight, $W$, of the loose roof layer that has to be carried.

$$\sum_{i-1}^{n} SB > W \qquad [2\text{-}4]$$

- The anchorage forces, $AF$, of the roof bolt system have to be greater than the weight of the loose roof layer.

$$\sum_{i-1}^{n} AF_i > W \qquad [2\text{-}5]$$

- Usually the support design incorporates a safety factor, $SF$.

$$\sum_{i-1}^{n} SB_i - SF.W > 0 \ and \ \sum_{i-1}^{n} AF - SF.W > 0 \qquad [2\text{-}6]$$

The number, $n$, of bolts/m$^2$ required to support a loose layer or layers of thickness, $t$, is given by:

$$n = SF\frac{\rho gt}{P_f} \qquad [2\text{-}7]$$

where, $SF$ = Safety Factor
$\rho$ = Density of suspended strata
$g$ = Gravitational acceleration
$P_f$ = Anchorage capacity

Suspension method is suitable in low-stress environments. However, horizontal forces can greatly increase the loads applied to roof bolts (Wright 1973; Fairhurst and Singh, 1974). Signer et al. (1993) found that measured loads on roof bolts are often twice what would be predicted by dead-weight design.

Beam theory has also been used since the 1980s in South African collieries in the design of roof bolt systems (Obert and Duvall, 1967; Wagner, 1985; van der Merwe, 1995; van der Merwe, 1998; van der Merwe and Madden, 2002). The parameters that govern the behaviour of gravity-loaded beams with clamped ends are as follows:

Maximum bending stress (MPa) $$\sigma_{xy} = \frac{\rho qL^2}{2t} \qquad [2\text{-}8]$$

Maximum shear stress (MPa) $$\sigma_{xy} = \frac{3\rho qL}{4} \qquad [2\text{-}9]$$

Maximum deflection (m) $$\sigma_{xy} = \frac{\rho qL^4}{32Et^2} \qquad [2\text{-}10]$$

where $L$ = roof span (width of roadway) (m)
$t$ = thickness of roof layer (m)
$\rho$ = density of suspended strata (kg/m$^3$)
$g$ = gravitational acceleration (m/s$^2$)
$E$ = Elastic Modulus (MPa)

In Australia, Frith (1998) proposed a model that is based on underground measurements and divides mine roofs into two classes:

- Static roof that is essentially self-supporting and requires minimum reinforcement; and
- Buckling roof that is thinly bedded and tends to fail layer by layer as a result of horizontal stress.

Frith (1998) proposed that the behaviour of the second type of roof can be explained by the basic structural engineering concept of the Euler buckling beam. There have been a number of trials of high-tension fully grouted bolts in Australia, and the results are reported to be positive. Unfortunately, the field evidence that has been presented to date has been largely anecdotal (Mark, 2000).

### 2.4.2 Field testing

The roof bolt design approach based on field testing was first developed in Australia (Gale, 1991; Gale and Fabjanczyk, 1993) and was largely adopted by the U.K. Code of Practice (Bigby, 1997).

The basic concept is that as individual roof beds become overstressed and fail, they force stresses higher into the roof, which can in turn fail more beds. Reinforcement aims to mobilise the frictional strength of failed roof beds in order to restrict the height and severity of failure in the roof. It involves measuring the loads developed in roof bolts during mining, together with a definition of the height and severity of roof deformation obtained from multipoint extensometers and sonic probe extensometers. On the basis of the measurements, Mark (2000) indicated that optimisation of the bolting design might include:

- Adjusting the bolt length so that adequate anchorage is achieved above the highest level in the roof where failure is occurring;
- Adjusting the bolt density and placement to maximise reinforcement where the roof needs it most; and
- Improving load transfer by reducing hole size, optimising bit type, or flushing the hole.

The results are considered valid for environments that are similar to the one studied. Significant changes in the geology or stress field require additional monitoring (Mark, 2000).

According to Altounyan and Taljaard (2001), the design based on field testing is based on two distinct stages:

- Detailed monitoring stations to provide design information; and
- Routine monitoring devices to measure and display roof movement.

The pull-out tests, roof monitoring using sonic probe extensometers, or tell-tales and instrumented bolts, are three main tools to determine:

- Changes in bolt load;
- Load transfer between the rock, the resin, and the bolt; and
- Roof deformations.

Design based on field testing incorporates short encapsulated pull tests, instrumented roof bolts and roof monitoring.

#### 2.4.2.1 Short encapsulated pull tests

The bond strength of a resin bonded roof bolt is a fundamental parameter determining the effectiveness. The stronger the bond, the shorter the anchorage zone of the bolt and the longer the full resistance zone over which the full bolt strength is available to resist roof movement (Mark et al., 2002).

With a mechanical anchor, the strength of the anchorage can be measured by pulling a standard installed bolt. With the resin-anchoring system, the anchorage provided by resin is related to the length of bond and the bond strength can easily exceed the strength of the steel (O'Connor et al., 2002). For this reason, a specially installed bolt with a shorter length of resin encapsulation is required in order to measure the anchorage properties of the resin anchor system rather than the strength of the bolt. This test has become known as the "short encapsulated pull test" (SEPT) and is an internationally recognised method of measuring the resin anchorage or bond properties of fully bonded roof bolts (UK Health and Safety Executive, 1996).

Standard SEPT equipment is shown in Figure 2-16.

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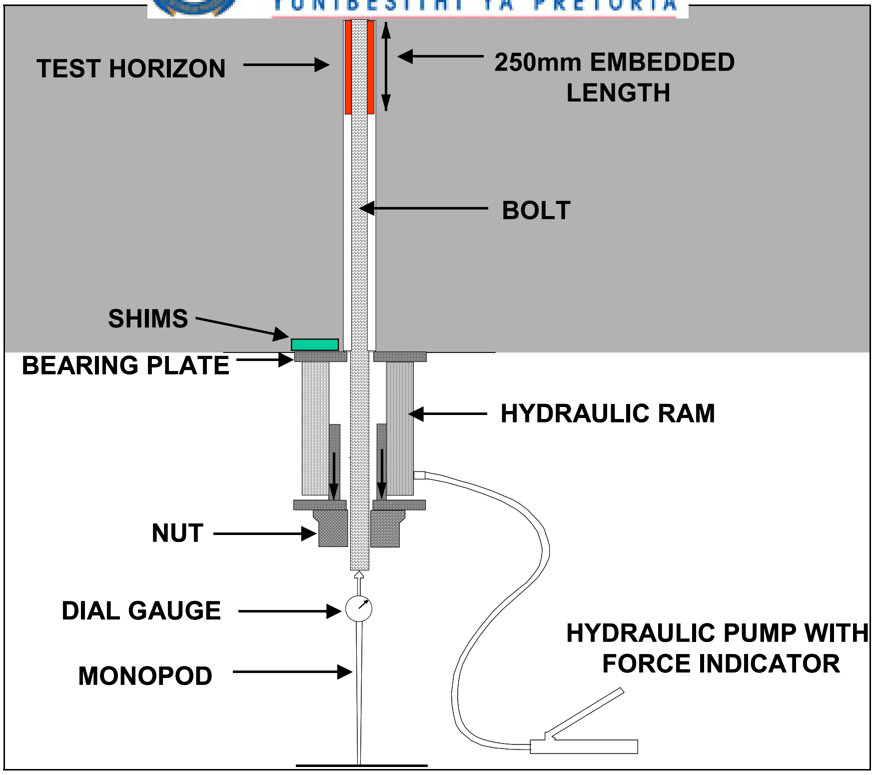


***Figure 2-16 Short encapsulated pull test equipment (after DMCIDC, 1996)***

The load-transfer capacity is a term equated to the effectiveness of the roof bolt in providing support to the rock strata. Serbousek and Signor (1987) defined it as the change in load with respect to distance along the roof bolt. Gray and Finlow-Bates (1998) defined it in terms of the maximum stress generated per unit area of the roof bolt. More effective support systems are characterised by high load-transfer capacity with high loads generated at small displacements.

Two models concerning the nature of load transfer with a fully encapsulated bolt have evolved over the past 30 years. One accounts for non-linear load transfer observed in pull tests undertaken in the laboratory and *in situ*. An alternative model accounts for linear load transfer, also observed in field studies but where load transfer was initiated through bed separation. Whitaker (1998) accounted for the two models as being due to differences in the method of loading the roof bolt. In a conventional pull test, an axial tensile load is applied at the free end of the grouted roof bolt usually being a hydraulic cylinder. At the same time, the resultant reactive force of the hydraulic cylinder induces a compressive load as it is made to press against the surface of surrounding rock.

Hagan (2003b) postulates that the more likely mechanism of loading a roof bolt in the field is caused by bed separation with a roof bolt being drawn in opposite directions by adjacent layers.

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Hagan (2003b) devised a laboratory test programme for investigating the effect of the loading method on a roof bolt under controlled laboratory conditions. Hagan (2003b) used two different methods to apply the load to the instrumented roof bolts. While the first method was intended to replicate a conventional pull test, the second method was intended to mirror the loading condition of a roof bolt subjected to bed separation. Hagan (2003b) found that: in the pull-test arrangement the rate of load transfer was non-linear; and in the bed separation arrangement, load transfer appeared to follow a linear reduction with distance. Hagan (2003b) suggested that caution should be exercised when results based on the pull-out test are interpreted, as it tends to overestimate the level of support that would actually be provided in supporting rock through load transfer and confinement.

In short encapsulated pull testing, the grip factor (bond strength), contact shear strength, and the system stiffness can be calculated as follows (Figure 2-17):

$$\text{Grip Factor } (GP) = \frac{F}{l} \quad \text{[kN/mm]} \tag{2-11}$$

$$\text{Contact Shear Strength, } (\tau) = \frac{F}{\pi d l} \quad \text{[MPa]} \tag{2-12}$$

$$\text{System Stiffness } (k) = \frac{\Delta F}{\Delta D} \quad \text{[kN/mm]} \tag{2-13}$$

where $F$ = Load to slippage (kN)

$\Delta F$ = Change in force (usually from 20 kN to 80 kN)

$\Delta D$ = Change in deformation (mm)

$l$ = Anchorage length (250 mm)

$d$ = Hole diameter (mm)

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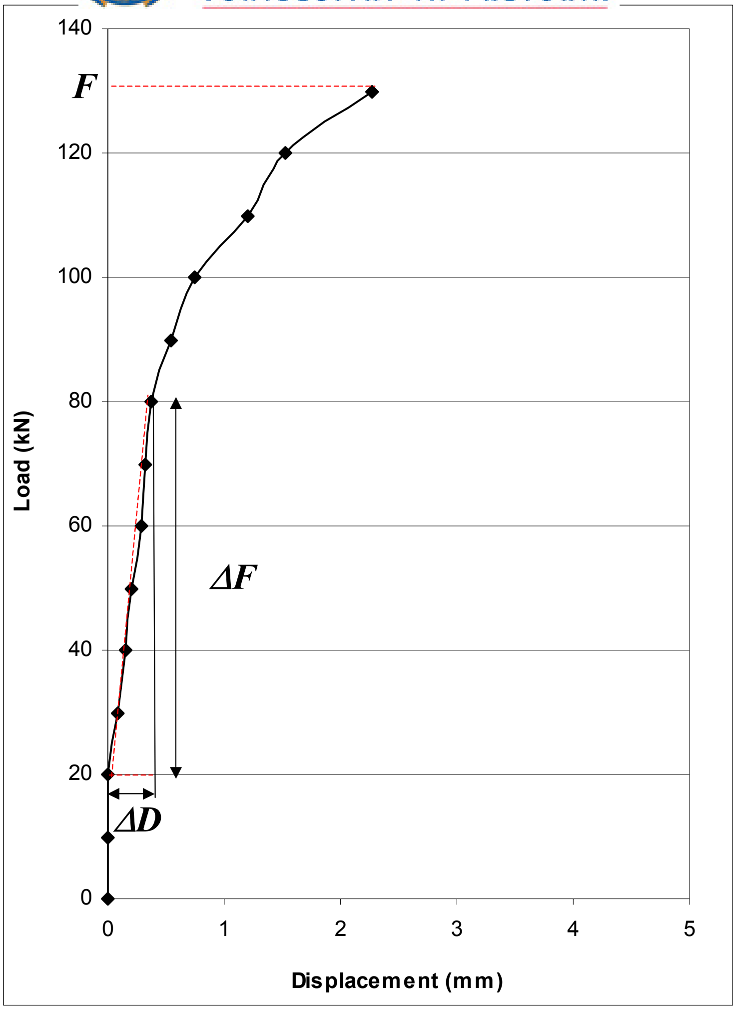


***Figure 2-17 A typical short encapsulated pull test result***

The key to using these relationships is that shear failure must take place between the resin-bolt, or resin-rock interface. In weak roof materials the resin-rock interface controls the failure mechanism. If the rock material is stronger, bond failure may occur on the resin-bolt interface. If tendon failure does not occur and the applied force exceeds the peak shear strength, the Equation [2-12] can be used to calculate the shear stress for the applied force, and system stiffness can be calculated from Equation [2-13] (Pile et al., 2003).

A good anchorage determined by short encapsulated pull tests is defined as one with minimum movement (high bond stiffness), where the anchorage capacity is equal to or slightly exceeds the bolt yield strength. A poor anchorage results in excessive movement and fails at lower loads than the bolt yield strength (Mark, 2004; Peng, 1986).

Biron and Arioglu (1985) state that the load distribution in the pull-out load of a bolt is determined by the ratio of elastic modulus of resin ($E_R$) to elastic modulus of roof rock ($E_{RR}$), *w*hen:

- $E_R / E_{RR} > 10$ load distribution is linear
- $E_R / E_{RR} < 10$ load distribution is non-linear

2.4.2.2 Instrumented bolt

An instrumented fully grouted bolt has pairs of strain gauges attached along its length (Figure 2-18). The strain along the bolt length can be measured, and the bolt load calculated by using the modulus of elasticity and the cross-sectional area of the bolt. The instrumented bolt can be used to measure the bolt loads sustained during different mining stages *in situ*. These bolt loads are compared with the yield load (Signer and Jones, 1990), allowing optimal design of the roof bolting system (bolt length and bolt spacing). Another way to design the roof bolting system is to calculate the total stress (axial and bending stresses) for every strain gauge (Signer et al., 1997).

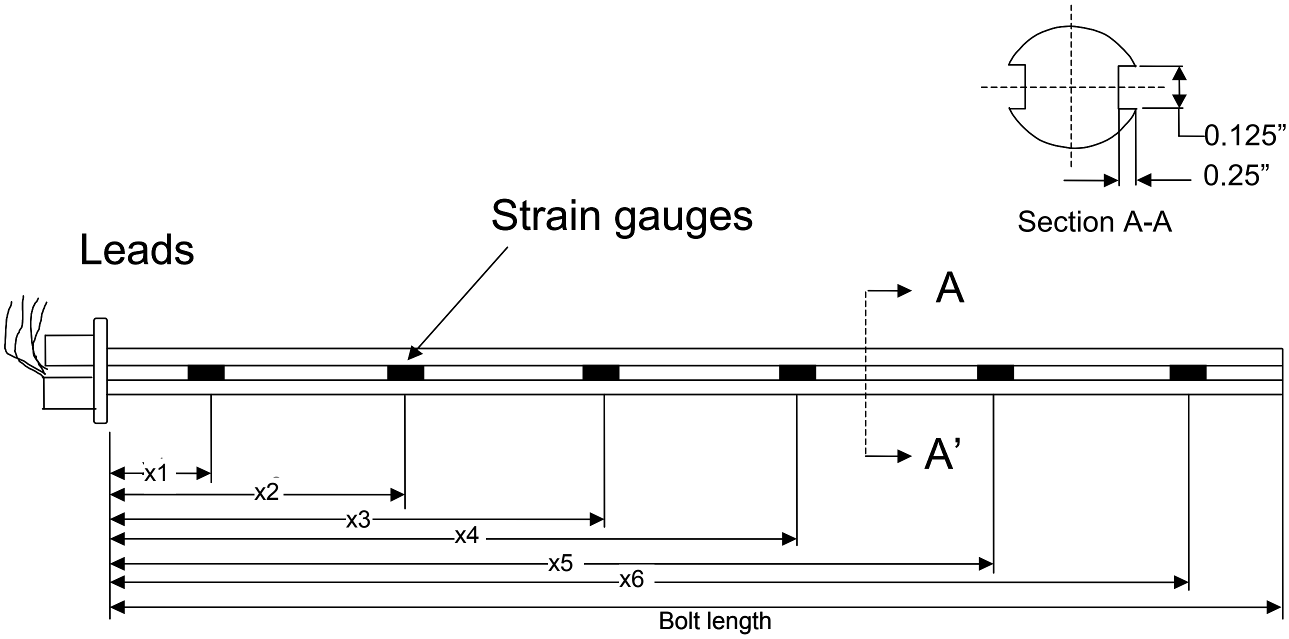


***Figure 2-18 Instrumented roof bolt (after Signer and Jones 1990)***

If the total stress (from field measurements) is greater than the maximum allowable stress, the following measures can be used to reduce the stress in the bolt:

1. Reducing the bolt spacing between rows;
2. Increasing the number of bolts per row; or
3. Increasing the diameter of the bolts.

Although the instrumented bolt can provide details about axial and bending load distribution along the bolt length, it has the following disadvantages (Signer, 1990):

1. The rebar is milled with a certain depth along each side, which will cause incorrect representation of the bolt area.
2. The maximum axial load or bending moment may be attained between the locations of the strain gauges and might not be measured.
3. The alignment of the strain gauges is critical to obtain good results.
4. The failure of strain gauges in some locations could be a result of wire failure or excessive loading, and can bias the readings towards one or more of the axial loading values (Signer and Lewis, 1998).

#### 2.4.2.3 Roof monitoring using sonic probe extensometer and tell-tales

Regardless of roof bolt design, failures are always possible. Often, an unstable area can be controlled with secondary support if the problem is detected in time (Yassien, 2003).

While routine monitoring of roof movements is only just becoming common practice in South Africa, it is enforced by regulations abroad. In the UK and Canada, tell-tales are required every 20 m in bolted roadways and in all intersections (Figure 2-19). The tell-tales have two movement indicators, one that shows displacement within the bolted height, and the other that shows movement above the bolts. Tell-tales are visible to everyone using the roadway, and the information provided by them can be recorded for later analysis (Altounyan et al. 1997).

Mark (2000) stated that the key to the effective use of monitoring is the determination of appropriate "action levels." In the UK, typical action levels are 25 mm within the bolted horizon and 10-25 mm above (Kent et al., 1999a). A survey of action levels in Australian mines, however, found no such uniformity (Mark, 2000). Some mines used total movement criteria; while others used rates of movement ranging from 1 to 10 mm per week (Mark, 1998). In the US, data is scarce, but action levels or "critical sag rates" are usually about 5 mm per week (Mark et al., 1994).

Often, roof monitoring can uncover a hidden geological factor that can then be accounted for in the design (Mark, 2000). For example, a back analysis of monitoring data from the Selby coalfields in the United Kingdom found that excessive roof movements occurred where entries were unfavourably oriented relative to the horizontal stress, and where the mudstone thickness exceeded 2.5 m (Kent et al., 1999b). At the Plateau Mine in Utah, Maleki et al. (1987) found that

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excessive sag rates correlated with the presence of a channel sandstone within 1.5 to 2.2 m of roof. A programme of test holes helped locate the sandstone and reduced the number of sagmeters required (Mark, 2000).

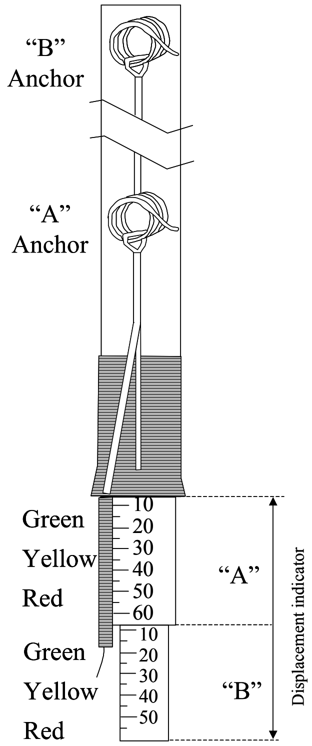


***Figure 2-19 A tell-tales (after Altounyan et al., 1997)***

### 2.4.3 Numerical modelling

Numerical methods of analysis are now widely used in rock engineering. The numerical methods used are listed in Figure 2-20.

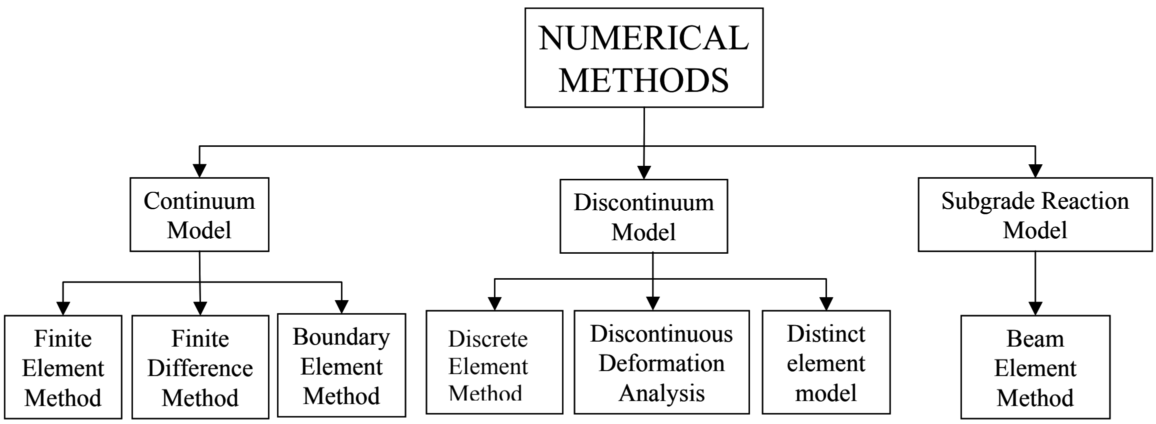


***Figure 2-20 Numerical methods in rock engineering***

For effective quantitative design using numerical models, three basic prerequisites must be met (Mark, 2000; Hayes and Altounyan, 1995; Gale and Fabjanczyk, 1993). These are set out in the paragraphs below.

Model: The model must be capable of replicating the behaviour of coal measure rock, which implies that it must be able to simulate the various failure modes and large deformations that typically occur.

Material properties and stress: Input rock mass properties must reflect both pre- and post-failure mechanics of the different rock layers encountered. *In situ* stress levels must be measured in the field.

Validation*:* To ensure that the model replicates underground behaviour, stresses and displacements must be measured. Important parameters include the magnitude and location of deformations, the distribution of bolt loads, and the behaviour of interfaces at the top of the pillar and within the roof.

Mark (2000) states that numerical models used in the US seldom meet any of these requirements.

Peng and Guo (1989) used a computer program consisting of a combined boundary-finite element method to analyse the stresses within roof reinforced by fully grouted roof bolts. The models incorporated weak bedding planes and horizontal stress. Different geological conditions and bolt patterns were used to develop the design criteria. They found that for a 6.1-m-wide roadway, the proper number of bolts varies from 4 to 6. To prevent failures associated with high horizontal stress, the number of bolts needs to be increased or the bolts need to be pre-tensioned during installation.

Theory describing roof bolt bond models and bolt models for inclusion in finite element and finite difference schemes are outlined by St. John and van Dillen (1983). Lorig (1985) re-iterates the theory specifically for explicit solution schemes and presents a number of empirical and analytic solutions for the shear response of bolts.

In recent work, roof bolts are effectively installed “over” an existing continuum mesh. The roof bolt nodes are therefore independent of the continuum degrees of freedom (i.e. the rock mass). Continuum elements and the roof bolt elements are connected through bond elements, thus permitting the simulation of grout, resin or friction coupling between the bar and the rock. Displacements from the continuum are transferred to the roof bolt system through these elements, and the resultant reactions are passed to the continuum as external loads. Roof bolt systems are constructed of interconnected layers of bond elements and axial structural elements. The constitutive models for both these types of elements are effectively one-dimensional and therefore are easily adjusted to account for any bond characteristic. In addition, there is the capacity for elements crossing discontinuities to generate reactions consistent with transverse shearing of roof bolts (Roberts, 2000).

#### 2.4.4 Roof support design based on geotechnical classification

The earliest reference to the use of rock mass classification for the design of tunnel support is by Terzaghi (1946) in which the rock loads, carried by steel sets, are estimated on the basis of a descriptive classification. Since Terzaghi (1946), many rock mass classification systems have been proposed, the most important of which are as follows:

- Lauffer (1958)
- Deere (1964): Rock Quality Designation, RQD
- Wickham et al. (1972): Rock Structure Rating (RSR – Concept)
- Bieniawski (1973): Geomechanics Classification, Rock Mass Rating
- Barton et al. (1974): Q- System
- Molinda and Mark (1994): Coal Mine Roof Rating (CMRR)
- Buddery and Oldroyd (1992): Impact Splitting Testing (IST)

Application of these systems in South African coal mines are discussed in detail in Chapter 5.0.

#### 2.4.5 Physical modelling

Physical modelling is a very useful tool for the design of underground roadways as it allows accurate measurement of bolt performance under controlled test conditions in the laboratory. Technically, however, it is difficult to ensure a consistent similitude ratio of geometry and material properties (Yassien, 2003).

An early attempt at a comprehensive design procedure was presented by Panek (1964). A series of scale model tests were conducted using limestone slabs to represent roof beds. The results were presented in the form of a monogram that related bed thickness and roof span to the required bolt length, tension, and pattern. Although Panek's monogram continues to be republished, it has not been used for practical design in decades (Fuller, 1999; Mark, 2000).

Several researchers have also used physical models to explore roof bolting performance (Fairhurst and Singh, 1974; Dunham, 1976; Gerdeen et al., 1979). All of these studies assumed that the roof was perfectly bedded, and it was consistently found that bolts located in the centre of the roadway added little to roof stability. In contrast, one model study of roof containing low-angle shears as well as bedding found that an evenly spaced pattern performs best (Mark, 1982).

By simulating a physical model for a roadway, Dunham (1976) studied the influence of bolt length, bolt spacing, bolt pattern, and inclination of the outer bolts on the stability of the roadway model. The fully grouted bolt was simulated by a 0.4 mm diameter silver wire, with resin was injected into the hole by syringes. It was found that the bolt length had a significant effect on the failure mode and that increasing the bolt length could increase the stability of the roadway. Moreover, the angled outer bolts create more stable conditions and reduce the diagonal shear cracks above the rib.

Another physical model was described by Tully (1987). It was found that the use of five 2.4 m bolts with two outer bolts inclined at 35 to 40° reduced the roof convergence.

Spann and Napier (1983) conducted a series of model tests to study and verify the beam-building concept in South Africa. Figure 2-21 shows the different roof bolting patterns that were modelled in the laboratory, and Figure 2-22 shows the effectiveness of the various patterns in controlling roof deflection. The effectiveness of roof bolts installed close to the roadway abutments in controlling shear movement, and hence beam deflection, is evident.

Spann and Napier (1983) concluded that the most important factor governing beam deflection is the location of the bolts in the beam, and that the best results are obtained if the bolts are installed close to the abutments of the beam.

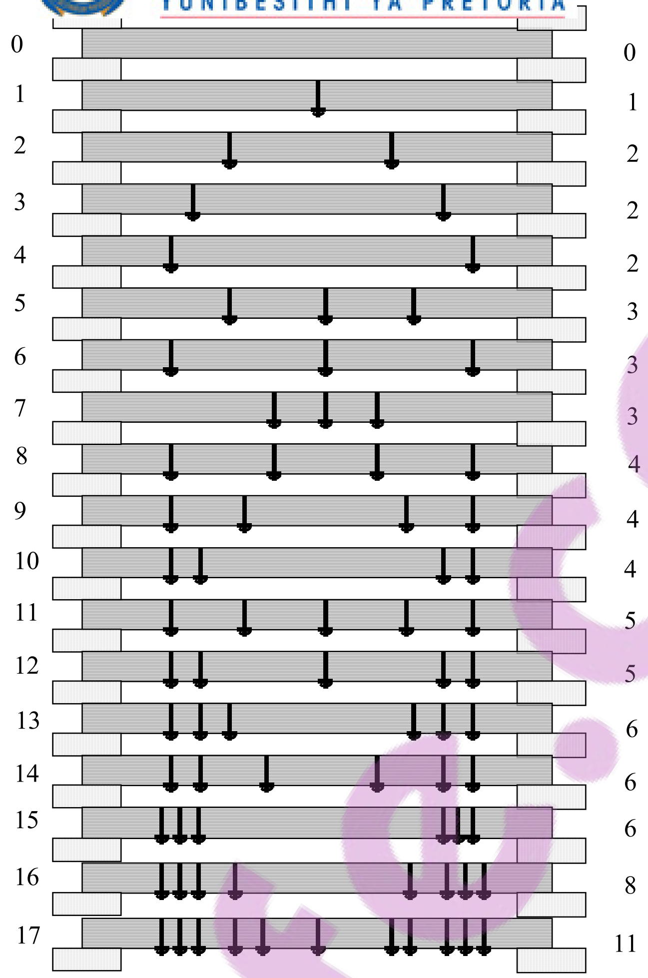


*Figure 2-21 Bolt pattern (after Spann and Napier, 1983)*

### 2.4.6 Probabilistic methods

Despite the fact that probabilistic design approaches have been widely used in civil and other engineering disciplines for more than two decades, only one study, by van der Merwe (1989), was conducted into the design of coal mine roof support systems using the probabilistic approach.

Van der Merwe (1989) determined the probability of failures in suspension roof support mechanisms to improve the decision-making process. The limitation of this study was that only two variables (thickness of the weak layer; thus the load on the system and the shear strength of contact between the resin, rock and bolt) were included in the study. There was no information on the variability of the other parameters, such as the bord width, distance between the bolts, strength of competent layer and the bolts etc. Nevertheless, in the early years of roofbolting in South Africa, van der Merwe (1989) showed that the probabilistic approach has

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many advantages over the deterministic approaches with respect to coal mine roof support design.

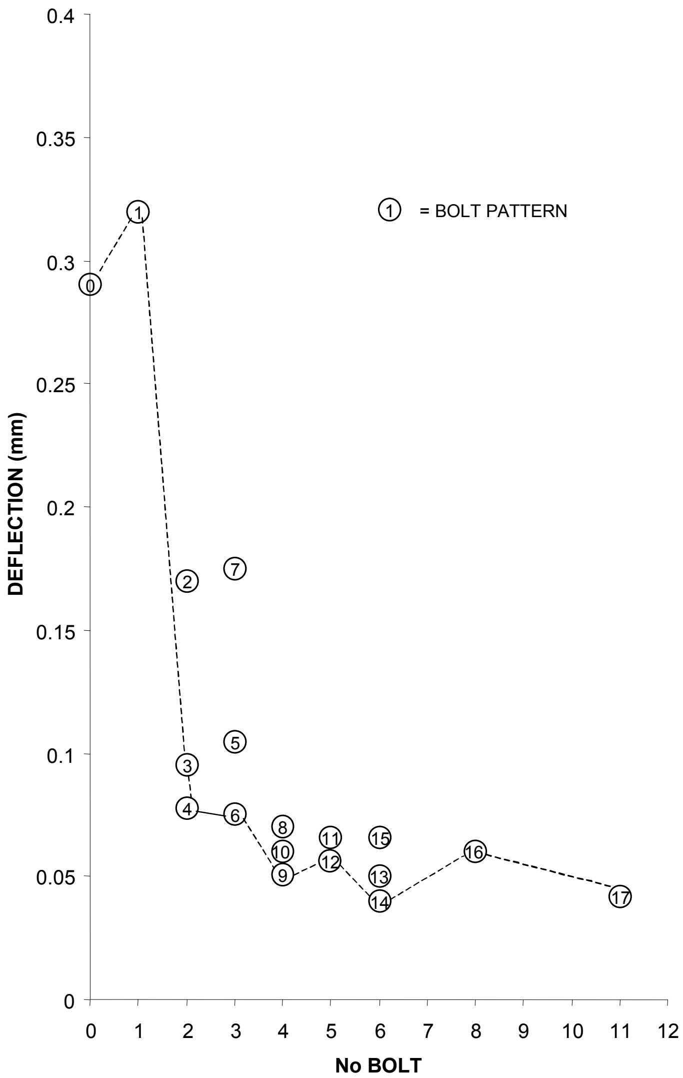


***Figure 2-22 Deflection compared to number of bolts (after Spann and Napier, 1983)***

## 2.5 Geometric parameters

### 2.5.1 Bolt length

The optimal roof bolt length depends on the support mechanism. Where bolts are merely acting as skin control, they may be as short as 900 mm (Minney and Munsamy, 1998). In the suspension mode, bolts should obtain at least 300 mm of anchorage in the solid strata (Mark, 2000). In the USA, federal regulations (30 CFR 75.204) require that when point-anchor bolts are used, test holes are to be drilled at least another 300 mm above the normal anchorage.

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Van der Merwe and Madden (2002) state that with resin bolts used in suspension mode, the longer the resin portion in the hole, the stronger the anchor. The bolt length must therefore be greater than the thickness of the laminated zone and have sufficient anchorage length above this zone to provide a strong enough anchor to suspend the laminations. The required strength of the anchor depends on the spacing of the bolts and the thickness of the laminated layer.

The required anchor length is determined by two methods in South African collieries. One is to use destructive pull tests to determine which minimum bond length will allow consistent failure of the tendon prior to anchor failure. The second method is to determine the mean shear strength of the bond, $\tau$, by means of short anchor tests. In these tests a short capsule (250 mm) is used, and the bolt is pulled to failure.

The bond length, $l_B$, is given by:

$$l_B = \frac{\delta^2 L_c}{D^2 - d^2} \qquad [2\text{-}14]$$

| | | | |
|---|---|---|---|
| where, | $\delta$ | = | capsule diameter (mm) |
| | $D$ | = | hole diameter (mm) |
| | $d$ | = | tendon diameter (mm) |
| | $L_c$ | = | capsule length (mm) |

The mean shear strength may then be calculated from:

$$\tau = \frac{P_f}{\pi D l_B} \qquad [2\text{-}15]$$

| | | | |
|---|---|---|---|
| where, | $\tau$ | = | mean shear strength (Pa) |
| | $P_f$ | = | yield load of bolt (N) |
| | $D$ | = | hole diameter (m) |
| | $L_c$ | = | capsule length (m) |

Once $\tau$ has been determined by short encapsulated pull tests, the calculation may be reversed in order for the required capsule length to be found through substituting $P_f$ for a design load. A suitable safety factor should also be used.

The proper bolt length is more difficult to determine in the beam-building mode. Van der Merwe and Madden (2002) suggest that the bolts must be longer than the thickness of the beam created, which is a function of road width, stress levels, etc.

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Several investigators have also studied the optimal length of the bolt that should be installed under various conditions. A summary of recommendations is given below. Note that $B$ is bord width (m) and $L$ is bolt length (m).

- Dejean and Raffoux (1976)

$$\begin{aligned} L_B &= 1 \text{ m (strong homogeneous rock)} \\ L_B &= (1/3 - 1/2)\, B \text{ (weak homogeneous rock)} \\ L_B &\geq 1.5 \text{ m (strong stratified rock)} \\ L_B &= (1/3 - 1/2)\, B \text{ (weak stratified rock)} \end{aligned} \qquad [2\text{-}16]$$

- Tincelin (1970)

$$\begin{aligned} L_B &> 1/3\, B \text{ (Roadways)} \\ L_B &\geq 1.25\,(1/3\text{ B}) \text{ (strong stratified rock)} \end{aligned} \qquad [2\text{-}17]$$

- Lang and Bischoff (1982)

$$L_B = B^{2/3} \qquad [2\text{-}18]$$

- Bieniawski (1987)

$$L_B = B/3 \qquad [2\text{-}19]$$

- Unal (1984)

$$L_B = \left[\frac{B}{2}\right]\left[100 - \frac{RMR}{100}\right] \qquad [2\text{-}20]$$

Where $RMR$ is the rock mass rating (Bieniawski, 1987) which ranges from 20 to 80 depending on roof rock conditions.

- Mark (2001)

$$L_B = 0.12(I_s)\log_{10}(3.225H)\left\{\frac{100 - CMRR}{100}\right\} \qquad [2\text{-}21]$$

Where: $I_s$ = Intersection span (average of the sum-of-the-diagonals, m)
$H$ = depth below surface (m)
$CMRR$ = Coal Mine Roof Rating

### 2.5.2 Bolt diameter

The yield capacity ($C$) of a roof bolt is normally determined by the bolt diameter ($D$) and the grade of the steel ($G$) (Mark, 2000):

$$C = \left(\frac{\pi}{4}\right) GD^2 \qquad [2\text{-}22]$$

This equation highlights that the yield strength of a bolt is proportional to the square of the diameter. In addition, as the bolt diameter increases, the stiffness of the bolt increases (see Section 2.7). Many authors argue in favour of greater bolt capacity to improve the effectiveness of roof bolts (Gale, 1991; Stankus and Peng, 1996). Higher capacity bolts are also capable of producing more confinement and promoting greater shear strength in the rock, and they may be pre-tensioned to higher loads (Mark, 2000).

Wullschlager and Natau (1983) analysed a finite element model to study the effect of changing the fully grouted bolt diameter on the load deformation behaviour of the bolt. The result showed that as the bolt diameter increases from 28.3 mm to 80 mm, the bolt stiffness increases.

Coats and Cochrane (1971) proposed the following formula for calculating the bolt diameter according to the yield strength of the steel:

$$R_{\max} = \sigma A \qquad [2\text{-}23]$$

$$P = \frac{R_{\max}}{SF} = 0.785 d^2 \frac{\sigma}{n} \qquad [2\text{-}24]$$

Where $R_{max}$ is the maximum bearing capacity of bolt; $P$ is the allowable axial load in the bolt in kN; $SF$ is the safety factor (chosen as 2-4); and $\sigma$ is the yield strength of the steel in kg/cm$^2$; $A$ is the bolt area in cm$^2$, $n$ is the number of bolts and $d$ is the bolt diameter in cm.

Mark (2001) suggests the following formula for determining the bolt pattern ($PRSUP$) and capacity:

$$PRSUP = \frac{L_B N_B C}{14.5\,(S_B W_e)} \qquad [2\text{-}25]$$

Where: $N_B$ = Number of bolts per row

$C$ = Capacity (kN)

$S_B$ = Spacing between rows of bolts (m)

$W_e$ = Road width (m)

Mark (2001) states that the suggested value of $PRSUP$ depends on the $CMRR$ and the depth of cover, as expressed in the following equations:

$$PRSUP = 15.5 - 0.23\ CMRR\ (shallow\ depth) \qquad [2\text{-}26]$$

$$PRSUP = 17.8 - 0.23\ CMRR\ (high\ and\ moderate\ depth) \qquad [2\text{-}27]$$

Where $CMRR$ is the Coal Mine Roof Rating

### 2.5.3 Bolt pattern

Lang and Bischoff (1982) found that the bolt spacing should satisfy the criterion that the ratio of bolt length to bolt spacing should be 1.5 in general, and a minimum of 2.0 in fractured rock. Bieniawski (1987) states that in coal mine roofs, this ratio should, in general, be 1.2.

In U.S. coal mines four bolts per row in 5.5 m to 6.1 m-wide roadways has become the near-standard and bolt spacing is constrained by law to a maximum of 1.5 m (Mark, 2000). However, according to Mark (2000), by international standards, 1.5 m bolt spacing is relatively light compared to the UK and Australian mines for beam building in high-stress conditions. In the UK, the minimum bolt density allowed by law is one bolt per square metre, and many Australian mines use similar bolt densities. In South Africa, however, there is no restriction for minimum bolt density. Therefore, the bolt spacing is greater, which has resulted in falls of ground in South African collieries (van der Merwe et al., 2001).

The field study reported by Maleki et al. (1994) found that increasing the bolt density reduces the average bolt load, while the total load remains approximately the same. Other researchers have found that when one side of the roadway suffers stress damage, bolts on that side sustain significantly higher loads (Mark and Barczak, 2000; Siddall and Gale et al., 1992). Additional bolts on the stress-damage side can help maintain overall stability (Maleki et al., 1994).

### 2.5.4 Annulus size

Karabin and Debevec (1978) states that the anchorage capacity of a roof bolt increases with roof bolt diameter; this holds true so long as the hole annulus or thickness of the resin between roof bolt and rock remains constant. For a constant annulus, increasing borehole diameter (and bolt diameter) increases both the maximum load-bearing capacity of the bolt and the shear strength of the resin/rock interface.

Snyder et al. (1979) argue that an increase in the borehole diameter must be accompanied by a commensurate increase in the diameter of the roof bolt, as this would otherwise lead to an increase in resin thickness. This would result in poor confinement of the resin, leading to a reduction in the shear strength of the bond.

Franklin and Woodfield (1971) found that when a 19 mm rebar was used, a resin annulus of 6.4 mm resulted in the strongest and most rigid anchorage system. Dunham (1976) suggests an optimal range of resin annulus of between 4 and 6 mm.

Wagner (1985) suggests that the bolt hole size should be a maximum of 6 to 8 mm greater than the nominal bolt diameter (3 to 4 mm annulus, which is defined as half of the difference between the bolt and hole diameters). This has been the design criterion in South Africa for many years. However, numerous tests have been conducted recently, which have shown that resin annulus is one of the critical variables that affect the bolt performance. The optimal difference between the diameter of the bolt and the diameter of the hole has been found to be no greater than 6 mm, giving an annulus of about 3 mm (Fairhurst and Singh, 1974; Karabin and Debevec, 1976; Ulrich et al., 1989).

Larger holes can result in poor resin mixing, a greater likelihood of "finger-gloving", and reduced load-transfer capability (Mark, 2000). Work reported by Fabjanczyk and Tarrant (1992) on roof bolt push tests showed a marked reduction in load-transfer performance of over 30 per cent with an increase in borehole diameter from 27 mm to 29 mm when a standard 22 mm roof bolt was used. Fabjanczyk and Tarrant (1992) suggest that the optimal borehole size is the smallest practical diameter when both bolt installation factors and resin viscosity are taken into account. Laboratory and field tests performed by Tadolini (1998), however, indicated that annuli ranging from 2.5 to 6.5 mm provided acceptable results in strong rock. Smaller holes can reduce the resin flow around the bolt, which may cause the loss of resin into bedding planes and vertical fractures in the rock mass (Campoli et al., 1999).

Hagan (2003a) conducted a series of laboratory tests to determine the effect of resin annulus on pull-out load. Mix-and-pour resin was used to avoid the effect of plastic packaging on the maximum pull-out load. It was concluded that there was an insignificant difference in the stiffness for resin annulus thicknesses of 2, 3 and 4 mm up to the maximum pull-out load. The results also showed that the lowest maximum pull-out load and post-failure stiffness were associated with the smallest annulus. Hagan (2003a) concluded that this may indicate the need for a minimum amount of resin for good bonding and load transfer between a roof bolt and rock.

In addition, it should be noted that smaller annuli (< 3.0 mm) may cause significant temperature rises during the mixing in the hole, which may accelerate setting of the resin, causing gellation before the determined setting time has expired.

## 2.6 Tensioned versus non-tensioned bolts

The choice of tensioned or non-tensioned bolts is one of the most discussed topics in roof bolting (Mark, 2000). A number of papers have been published on this topic in Australia and the

US. The issue is complicated, as there are three possible systems: fully grouted non-tensioned, fully grouted tensioned, and point-anchor tensioned.

Peng (1998) states that resin-assisted point-anchor tensioned bolts can be used to clamp thinly laminated roof beds into a thick beam that is more resistant to bending. In addition, Stankus and Peng (1996) state that by increasing frictional resistance along bedding planes, roof sag and deflection are minimised and that lateral movement due to horizontal stress is less likely. Tensioned bolts are also more efficient, because a stronger beam can be built with the same bolt by applying a larger installed load (Mark, 2000).

Frith and Thomas (1998) and van der Merwe and Madden (2002) advocate pre-tensioning fully grouted bolts using two-stage resins and special hardware. Frith and Thomas (1998) argue that active pre-loads modify roof behaviour by dramatically reducing bed separation and delimitations in the immediate 0.5 to 0.8 m of roof. In addition, Frith and Thomas (1998) state that the key reason why tension works can be better understood if the roof is seen as an Euler buckling beam. In the presence of a pre-tensioned beam, small vertically applied loads have less potential to cause instability.

Gray and Finlow-Bates (1998) found that non-tensioned, fully grouted bolts with good load-transfer characteristics may be just as effective. It is argued that a preload of 100 kN results in a confining stress of only 70 kPa within the immediate roof, which is small compared to *in situ* horizontal stresses, which are at least 10 times greater. Others have observed that in field measurements, resin bolts have quickly achieved loads that are even greater than those on nearby point-anchor bolts (Mark et al., 2000). McHugh and Signer (1999) showed that, in laboratory tests, pre-tensioning fully grouted bolts did little to strengthen rock joints.

Fuller (1999) concludes that "the generally positive results of field trials indicates that pre-tensioning, when combined with full bonding of bolts, provides the maximum strata reinforcement".

Unfortunately, direct comparisons of the three systems are rare (Mark, 2000). Anecdotal evidence is often cited, sometimes from situations where bolt length, capacity and pre-tension were changed (Stankus, 1991). There is a consensus that large preloads are not necessary for resin bolts to function effectively in the suspension mode (Peng, 1998; Frith and Thomas, 1998; Maleki, 1992), but more research is suggested for broader conclusions to be drawn.

While plate loads may be typically 30 to 50 kN in South African collieries, Singer (1990) measured plate loads of approximately 11 kN. Plate loads can increase by a factor of ten or

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more in highly deformed ground (Tadolini and Ulrich, 1986). Plate loads were also measured in South Africa (Canbulat et al., 2003), as a function of time. Figure 2-23 indicates that the load on the plate reduces over time.

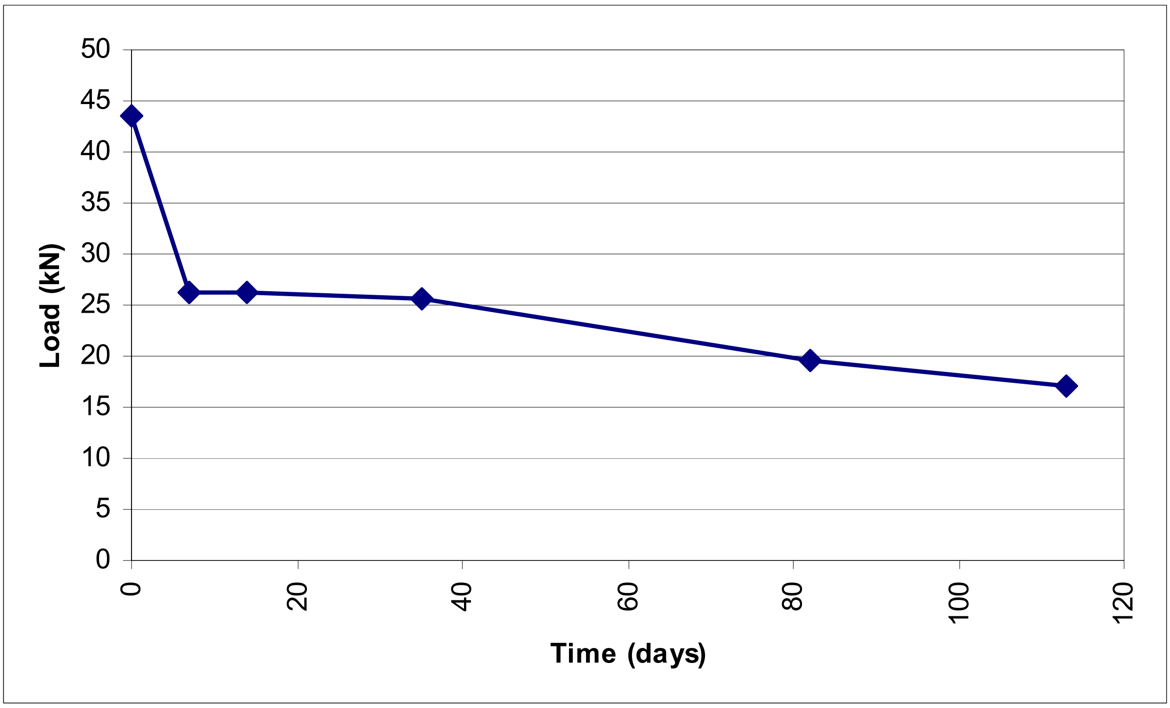


***Figure 2-23 A typical plate load versus time in South African collieries (after Canbulat et al., 2003)***

## 2.7 Stiffness of roof support

Stiffness is a measure of how quickly a support develops load-carrying capacity in response to roof strata dilation (Mark, 2000). Stiffer supports will develop capacity over less displacement than softer supports.

Stiffness ($K$) is a function of the area ($A$), material modulus of elasticity ($E$), and the length of the support ($L$):

$$K = \frac{A\,E}{L} \qquad [2\text{-}28]$$

This equation indicates that the stiffness increases with increasing area (bolt diameter) and material modulus (steel modulus) and decreases with increasing length. It should be noted that, with a conventional point-anchor mechanical roof bolt, the bolt is anchored only at the top, and the “free length” of the bolt is the entire length of the bolt less the anchored length. In full-

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column resin bolts, the "free length" of the bolt is less, and the full-column roof bolts hence provide stiffer support than mechanical bolts (Mark, 2000).

## 2.8 Intersection support

Intersections are particularly susceptible to strata control problems as a result of inherently wide roof spans and resulting induced stress. This situation is accentuated in the presence of high horizontal stresses. As a result many authors have investigated this problem area (Gercek, 1982; Hanna and Conover, 1988; Vervoort, 1990; Molinda et al., 1998; Canbulat and Jack, 1998; Mark, 2001; Zhang, 2003; van der Merwe et al., 2001, van der Merwe and Madden, 2002).

Vervoort (1990) investigated the fall of ground (FOG) fatalities in South African collieries. It was found that 43.4 per cent of all FOG fatalities for the period 1970 to 1988 occurred in intersections. Further analyses of FOG fatalities carried out by Canbulat and Jack (1998) also showed that the majority of FOG fatalities (36 per cent) for the period covering 1989 to 1995 occurred in intersections. Van der Merwe et al. (2001) also conducted a study into the causes of FOG in South African collieries. Again, it was found that the majority of all roof falls occurred at intersections, which were responsible for 66 per cent of the 182 falls of ground investigated. Note that there was no mention in these studies of whether the intersections were supported or not.

Van der Merwe et al. (2001) state that intersections account for approximately 30 per cent of the total exposed roof, which means that the risk of a roof fall in an intersection is more than four times greater than in a roadway. According to Molinda et al. (1998), approximately 71 per cent of all FOG occurred in intersections, indicating that the roof fall rate in the US is eight to ten times greater in intersections than in roadways.

Studies have shown that intersection stability is a function of rock quality and the ratio of horizontal stress to vertical stress (Molinda et al., 1998; Gercek, 1982; Unal, 1984). The following steps were recommended by various authors for reducing the risk of failure in intersections:

- Roof control plans should be developed that specify the maximum spans that are allowed (Molinda et al., 1998; van der Merwe and Madden, 2002).
- Mining sequence should be designed to limit the number, location, and size of splits, and not to orient splits at critical angles to the principal horizontal stress direction (Molinda et al., 1998, Hanna and Conover, 1988).

- Bolt length and bolt density in intersection corners near the ribsides should be increased (Hanna and Conover, 1988; Zhang, 2003).
- Splits should be holed only into supported intersections (Minney and Munsamy, 1998).

On the other hand, Molinda et al. (1998) found that replacing four-way intersections with three-way intersections may be not an effective control technique in terms of roof stability.

Current practice for supporting intersections is to use the same roof bolt design as for roadways, seldom with additional supports. Local experience has often determined additional support in intersections. In order to support the intersections efficiently, a better understanding of rock behaviour in intersections is required. The influence of different strata conditions on this behaviour needs to be determined so that better support design and installation rules can be facilitated.

## 2.9 Discussion and conclusions

Since the introduction of mechanical bolts in the 1940s, the amount of research into the understanding of the behaviour of roof bolts has been significant. Today, almost all coal mine roofs are supported with roof bolts in South Africa.

In the early years, the design of roof bolt patterns was based on local experience and the judgement of mining personnel. The suspension mechanism was the most easily understood and most widely used roof bolting mechanism. However, significant advances have been made over the last 20 years, in particular, the development of resin anchors, tendon elements, and installation hardware. These advances have resulted in an increase in the use of full column resin bolts.

The design of roof bolt patterns has also been improved, and four main rock reinforcement techniques have been developed: simple skin control, beam building, suspension, and keying. The geology and the stress levels determine the appropriate mechanism for a particular application.

The importance of tensioning of roof bolts remains a subject of controversy. As will be seen in the following chapters, the critical roof deformations in South African collieries are relatively small. Therefore, tensioned roof bolts are beneficial in that they allow less roof deformation to take place after the support has been installed. However, if the bolting system is stiff enough, tensioning may not be required.

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Although there have been many studies into the support of intersections, a better understanding of rock behaviour in intersections is required.

Numerical models are useful in understanding roof and roof bolt behaviour; however, extensive laboratory studies are required for determining the input parameters for site specific conditions. The Australian technique, subsequently adapted in the UK, has proven that numerical modelling can be used to back analyse underground scenarios. Once the model is calibrated, then the results obtained from the numerical models can be used for design. No attempt has been made to develop a generic numerical model to be used in the design of roof support systems.

The selection of roof bolt type for different geological environments is well documented. However, the changing conditions underground must also be determined and the design and the support system have to be modified accordingly. Widespread instrumentation and vigilant visual observations are important for ensuring safety and stability in coal mines.

While the effect of roof bolt diameter on support performance is well understood, there is still controversy over the length of the roof bolts. It has been shown by Molinda et al. (2000) that the probability of roof failures increases with decreasing bolt length. Since skin failures (< 0.5 m thick) are more common in South Africa than larger roof falls (Canbulat and Jack, 1998, van der Merwe and Madden, 2002), short roof bolts for skin control may make up part of an effective support system.

Although, roof bolting has probably been the most researched aspect of coal mining, FOG still remains the major cause of fatalities in South Africa. There are no commonly accepted design approaches available for underground coal mines. Roof bolts were found to behave differently under different loading conditions, emphasising the importance of understanding the interaction between the roof bolts and the rock mass.

In conclusion, this review showed that the most important key to the design of a roof support system is a better understanding of roof behaviour and uncertainties that can be encountered during extraction. Different support design methodologies have been developed based on rock mass classification techniques, numerical modelling, instrumentation and monitoring and physical modelling. However, majority of these techniques are based on deterministic approaches using localised information and no significant attempt has been made to develop a probabilistic design methodology, which takes into account the natural variations exist within the rock mass and the mining process. It is therefore concluded that the probabilistic approach is a step forward in the design of coal mine roof support systems.

In the following Chapters of this thesis, an attempt will be made to understand the roof and support behaviour in South African collieries through *in situ* monitoring and also a probabilistic model, which describes both the strength and the load acting on rock, will be defined using the stochastic modelling technique.

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# Chapter 3.0

# Underground monitoring of roof and support behaviour

## 3.1 Introduction

One of the most important prerequisite in the design of a support system is to understand the roof and support behaviour in different geotechnical environments. An extensive monitoring programme was therefore undertaken in order to establish the behaviour and the interaction between the support units and the roof. Critical deformations beyond which the roof fails will occur was also investigated.

A total of 29 sites at five collieries were monitored using sonic probe extensometers and in order to cover as much of the roof strata as possible, and avoid losing what could in time turn out to be valuable information, the full string of 21 anchors with the top anchor at approximately 7.3 m was installed at all the monitoring sites.

To process the monitoring data as quickly and efficiently as possibly, a customised program was written as part of this study, culminating in an easy to understand set of graphic results. The basic function of this program is to compare all subsequent sets of readings with the original set and produce displacement-with-time graphs. Various modifications and improvements were introduced to include the option of producing velocity and acceleration graphs to assist with the interpretation of the results.

## 3.2 Underground monitoring procedure

In this monitoring programme sonic probe extensometer is utilised. The sonic probe extensometer system is a sophisticated electronic device. It generates a pulse that travels at the speed of sound, and is able to accurately determine the distance between magnetic fields, set up by magnets which are integral to the extensometer anchors.

The cylindrical magnetic anchors are locked in place at predetermined locations in a borehole and have a plastic tube inserted through their centres. This tube acts as a guide for a flexible probe that is then inserted through the entire string of anchors. The readout unit is connected to the probe and the distances between the magnetic fields are individually displayed and manually recorded.

In order to record all the information relevant to roof strata deformation prior to the installation of any roof support, would necessitate the installation of instrumentation a few metres ahead of the face. Since this is clearly not possible the next best scenario is to install the instrumentation at the face. However, due to practicalities such as not working under unsupported roof and the limitations on how close the roofbolters can get to the face, it is not usually possible to drill closer than about 0.5 m from the face. This results in the monitoring hole being in or close to the last row of support.

Drill bit sizes, resin quantities and support types and lengths were also monitored. In the underground situation the quality of roof support installation is dependent on a number of factors. With resin bonded bolts the bond length and quality are dependent on the actual average hole diameter, the overdrilling of holes and deviations from the recommended resin spin and hold times. It was not practical or possible to monitor or control the support installation at the monitoring sites. The support performance monitored is therefore a true representation of the support systems as installed underground and includes any effects linked to imperfections in the installation of the support.

At the monitoring site, close to the face, and situated in the middle of the advancing roadway, an 8.0 m deep hole was drilled vertically with a roofbolter into the roof and reamed out to 50 mm in diameter to accommodate the sonic probe magnetic anchors. Although most of the drilling process was carried out with water flushing, the final reaming of the hole is done dry, as the modified custom made reaming bits cannot accommodate water channelling. The hole was cleaned by inserting a water hose to the top or by spinning one of the smaller drill bits up the hole with the water switched on. A petroscope was then inserted into the hole and the lower 2.5 m was examined to detect the presence of any open laminations or fractures. However, final reaming of the holes to enlarge the hole by a few millimetres was carried out dry. During this process, the moisture left in the hole by the original wet drilling mixed with the powdered coal duff and form a paste that was then smeared into any openings by the reaming process. Therefore, the petroscope monitoring of the holes did not result in reliable information and was taken out of the monitoring programme.

A full string of 21 anchors was then installed in each hole at predetermined intervals (approximately 250 mm apart) using a set of installation rods. The top anchor, the first to be installed, is placed at approximately 7.3 m. An extra anchor that does not have a magnet fitted is installed in front of the last anchor, a short distance from the collar of the hole. This is a prerequisite in a vertical hole and is used to suspend the sonic probe to prevent it moving during the reading process.

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Depending on the mining method and speed of face advance, the time lapse between further sets of readings varied from hours to days apart. In a typical development section underground three or four sites close to the centre of the panel were monitored. Where possible, the sites included both roadways and intersections to be able to evaluate and compare the strata behaviour and support performance in the two different locations. Prior to any development of the intersection taking place, the instrumented hole was positioned at the face so as to be as close as possible to the centre of the proposed intersection.

Survey levelling was used in conjunction with the sonic probe to assist in assessing the accuracy of the probe. The relative displacement measured between points anchored at 0.1 m in from the roof skin and at an elevation of approximately 1.8 m should ideally be compared against displacements measured between anchors at similar elevations by the sonic probe. However, at most of the monitoring sites where levelling was implemented, all the roof displacements took place within 1.8 m of the immediate roof. The levelling results have therefore been compared with the “total relaxation” measured by the sonic probe. The total relaxation is the overall displacement between a stable elevation in the roof and the anchor closest to the roof skin. In the five cases (Colliery D area 2) where displacements occurred up to 2.5 m into the roof, a note concerning the comparative probe and levelling displacement values has been included in the appropriate figures. These values have also been included in Table 3-1 where direct comparisons can be made between the sonic probe results and all the sites where back up levelling was successfully implemented.

In some cases it was not possible to make use of the survey levelling backup system due to factors such as the dip of the seam and the mining method and sequence. Levelling monitoring points that were damaged during the monitoring period were excluded from the results. Levelling backup was successfully implemented at approximately half the monitoring sites. The survey levelling results were included in the sonic probe displacement graphs. In excess of 90 per cent of the cases, the levelling results recorded similar or higher values than those of the sonic probe. A higher value levelling result is perfectly acceptable since the levelling skin anchor is usually about 0.1 m closer to the roof skin than the lowest sonic probe anchor, which is usually placed 0.2 m into the roof. Any displacement that occurs between their respective elevations would only be recorded by the levelling results.

## 3.3 Processing of information

The initial readings were taken as soon as the installation was completed. These comprise a minimum of three sets which were screened for any obvious anomalies or booking errors. They were then entered into the program where they were averaged, and the calculations carried out

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to produce the graphic results necessary for interpretation. All the subsequent sets of readings were treated in a similar manner with the program comparing them to the first (datum) set of readings from which the displacements were calculated.

The original displacement graphs included all the anchors in the hole up to the 7.3 m elevation. However since the main focus of the investigation was in the vicinity of the support horizon all the support performance graphs have been cropped at the 2.5 m elevation. This does not infer that displacements above the 2.5 m elevation were discarded or ignored.

Included alongside the 2.5 m vertical axis on each graph is a shaded block representing the section of strata column under investigation. The patterns within the block represent the approximate location of the different strata types, typically sandstone, shale and coal. These patterns are included and labelled in Figure 3-1. The stratigraphic column included with each individual displacement graph is representative of the area under investigation.

Although in some cases as many as 15 site visits were carried out and sonic probe readings taken, individual composite graphs have been limited to a maximum of five sets of readings for reasons of clarity.

In order to present the results of the individual site investigations in a simple and efficient manner, a graphic classification system was used. An explanation of this system and the relevance of other information included with it are given in Figure 3-1.

Although the displacements usually start at the roof skin and are evident for some distance into the roof, the section of the strata column under investigation does not extend down to the roof skin. The reason for is that the bottom magnetic anchor of the anchor string has to be approximately 0.2 m into the roof to allow the dummy anchor, used as a suspension point for the sonic probe, to be installed in front of it.

The displacements recorded by the final set of sonic probe readings taken at a particular site are transferred to the strata column. Here they are shown as individual lines approximately midway between the anchors from which each relative displacement value was calculated.

***Table 3-1 Sonic probe, levelling and stable roof elevation results***

| Colliery and Roof strata | Mining method | Support type | Monitoring Position | Probe relaxation (mm) | | | Levelling (mm) | Stable roof elevation | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | **Total** | **Averages** | **1.8 m - skin** | **1.8 m - skin** | **(m)** | **Averages** |
| A | C M | Full column | Roadway | 2.5 | | 2.5 | | 1.1 | |
| Shale | | resin | **Roadway** | 0.2 | 1.3 | 0.0 | | 0.0 | 0.6 |
| B | D&B | Partial | Roadway | 2.0 | | 1.2 | | 2.2 | |
| Coal roof | | column | Roadway | 0.5 | | 0.5 | 1.0 | 2.0 | |
| 0.3 m | | resin | Roadway | 2.5 | | 2.3 | 2.5 | 2.0 | |
| shale | | | **Roadway** | 3.0 | 2.0 | 3.0 | | 2.2 | 2.1 |
| then coal | | | *Intersection* | *4.0* | | *3.8* | *4.2* | *2.0* | |
| above | | | ***Intersection*** | *6.0* | *5.0* | *6.0* | | *2.0* | *2.0* |
| C | D&B | Mechanical | Roadway | 1.5 | | 1.5 | 1.5 | 2.5 | |
| 0.3m coal | | end | **Roadway** | 0.5 | 1.0 | 0.5 | | 1.9 | 2.2 |
| with shale | | anchored | *Intersection* | *3.5* | | *3.5* | *5.7* | *1.9* | |
| above | | | **Intersection** | *1.0* | *2.3* | *1.0* | | *2.2* | *2.1* |
| | D&B | Full column | Roadway | 0.5 | | 0.5 | 1.0 | 1.4 | |
| | | resin | **Roadway** | 1.0 | 0.8 | 1.0 | 2.0 | 1.1 | 1.3 |
| | Area 1 | | *Intersection* | *6.5* | | *6.5* | *7.0* | *1.4* | |
| D | | | ***Intersection*** | *2.5* | *4.5* | *2.5* | *4.0* | *1.9* | *1.7* |
| Inter | C M | Partial | **Roadway** | 2.5 | 2.5 | 2.0 | 3.7 | 2.0 | 2.0 |
| laminated | | column | *Intersection* | *2.0* | | *2.0* | | *1.8* | |
| sandstone | Area 2 | resin | *Intersection* | *12.0* | | *11.5* | *11.2* | *2.5* | |
| and shale | | | *Intersection* | *12.0* | | *9.5* | *12.5* | *2.5* | |
| | | | **Intersection** | *6.0* | *8.0* | *5.0* | *10.0* | *1.9* | *2.2* |
| | C M | Full column | Roadway | 1.0 | | 1.0 | | 0.8 | |
| | | resin | **Roadway** | 0.5 | 0.8 | 0.5 | | 1.0 | 0.9 |
| | Area 3 | | **Intersection** | *1.0* | *1.0* | *1.0* | | *1.2* | *1.2* |
| E | C M | Full column | Roadway | 1.0 | | 1.0 | | 0.5 | |
| Sandstone | | resin | **Roadway** x 4 | 0.2 | 0.2 | 0.0 | | 0.0 | 0.1 |

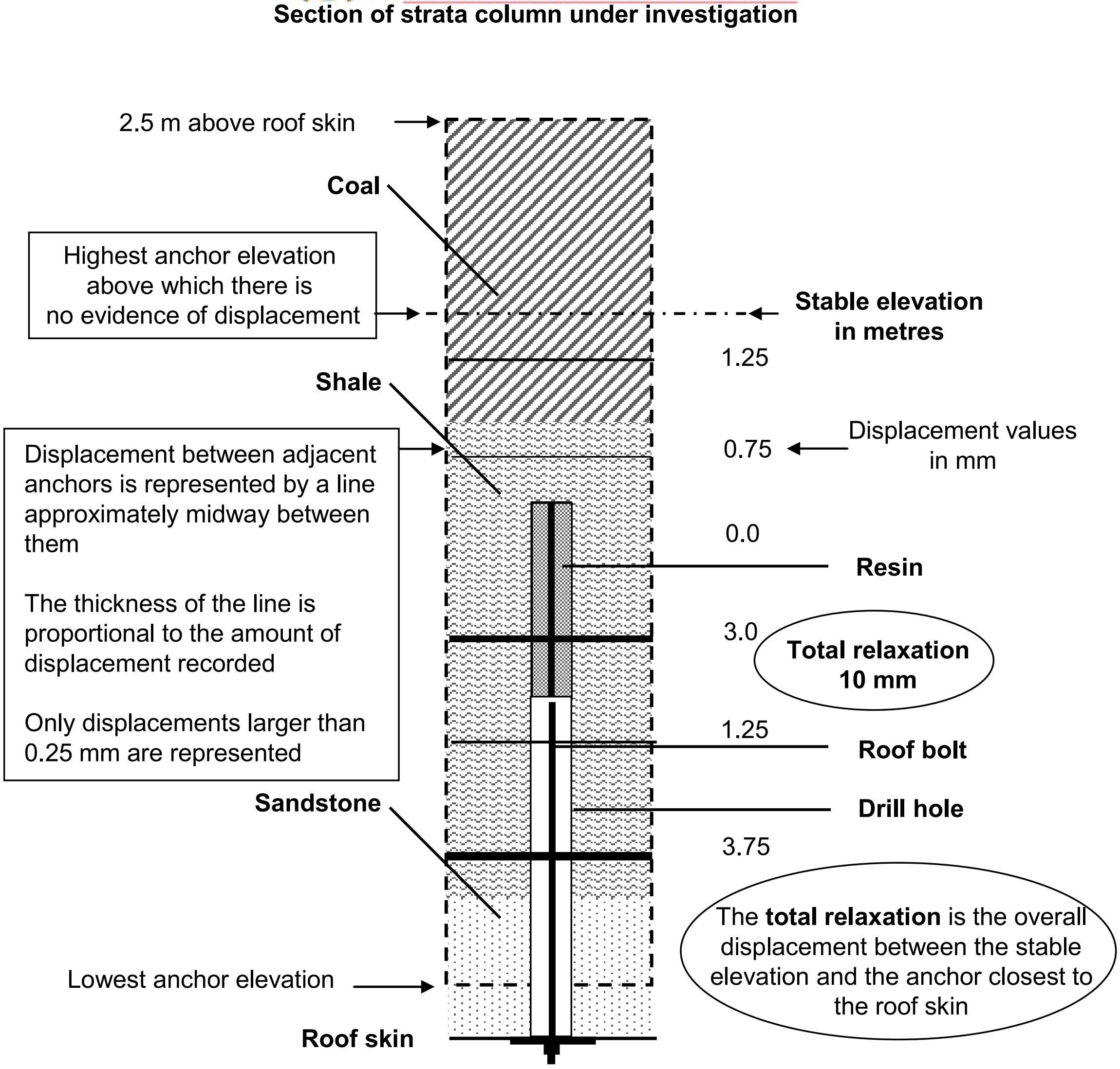


***Figure 3-1 Graphic representation and explanation of a typical geological profile, support type and final roof strata behaviour***

In order to establish a uniform approach to assist in simplifying the interpretations, the following criteria were introduced:

- Only readings outside the accepted error band were accepted.
- Differential displacements between adjacent anchors had to exceed 0.5 mm to be considered, except in the case of a trend involving three or more anchors where displacements down to 0.25 mm were included.

Displacements of 0.25 mm and larger are therefore represented by a line. In order to emphasis the different magnitudes of the various displacement zones, each line has been designated an appropriate thickness proportional to the value. These lines represent the total displacement recorded within the zone (between the two anchors) and do not infer that all the displacement

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took place at one particular elevation or parting plane; they are primarily an indication of relative magnitudes.

In Figure 3-1, to assist in explaining this concept, the anchor string showing individual anchor elevations is included. Alongside each displacement line the individual displacement values have been recorded. Where no displacement was observed, a zero value (0.0) is evident, as is the lack of a displacement line. The method used to indicate a negative displacement is also indicated. The anchor string and displacement values are included in Figure 3-1 primarily to assist with the explanation. They are not recorded in the graphic presentations of the individual monitoring site figures, as this information is already present in a slightly different form in the sonic probe graph.

To assist in assessing the effectiveness of the various roof support systems, a single support member is also included as part of the shaded strata column block alongside each sonic probe graph. The length of both the support member and the anchoring mechanism is drawn in at the same scale as the vertical axis of the sonic probe graph. A partial column resin anchored bolt is shown in Figure 3-1.

The roof displacements measured by the sonic probe are superimposed on the relevant roof support member for comparison purposes. This does not necessarily infer that these displacements are occurring in or at the support tendon hole, particularly where the hole is full of resin. The sonic probe hole varied between 0.3 to 1.0 m away from the closest support tendon hole.

The anchor height above which no displacements were recorded in a strata column is indicated as the 'stable elevation'. In cases where some doubt exists it may be referred to as the 'estimated stable elevation'. The 'total relaxation' value indicates the overall displacement between the stable elevation and the bottom anchor in the string.

Included with each displacement graph is a list of notes covering the monitoring site position, layout and mining method as well as a description of the roof strata and support system installed.

## 3.4 Colliery 'A'

Two sites, both in the same roadway 43 m apart, were monitored at Colliery 'A'. Guttering on one side of the roof/sidewall contact appeared to develop one or two pillars back from the face in roadways travelling in the same direction as the roadway where the monitoring sites were installed. Although in some cases the guttering was semi continuous for two or three pillars, its general appearance appeared to be random in nature. A number of intersections had collapsed and some roadways had been barricaded off due to dangerous roof conditions, usually associated with the guttering. Petroscope holes, drilled into the roadway roof where there were obvious roof problems, detected displacements up to a height of 1.6 m into the roof. Within this zone a number of openings in excess of 10 mm were observed.

The colliery was situated in the Vereeniging Coalfield mining the 2b Seam at a depth of 70 to 80 m with a mining height of 3.0 m. Mining was carried out using a continuous miner with onboard roofbolters. The roof was shale supported by 2.1 m long AX bars 21 mm in diameter with full column resin in a 25 mm diameter hole. The 5.0 m wide roadways were supported with five to six bolts per row with 'W' straps. The rows were 1.0 m apart.

The monitoring results from the two holes are presented in Figure 3-2 and Figure 3-3. The monitoring hole installation positions relative to the face were governed by how close the continuous miner with its onboard roofbolters could get to the face. Face advances in excess of 60 m took place during the two month monitoring period. At site 1 (Figure 3-2) displacements were only recorded below the 1.1 m elevation. The total relaxation of the lowest anchor was 2.5 mm bearing in mind that this displacement is relative to the stable elevation. At site 2 (Figure 3-3) no displacements were detected. Unfortunately, it was not possible to install the survey levelling backup system at either site.

While the failures were visually observed in other parts of the section, there was no visual evidence at either of the two sites to indicate the presence of a high horizontal stress regime. These results clearly illustrated the site specific nature of each monitoring site. The support system installed was adequate to control the shale roof in the regions where it was not subjected to the buckling effects of a high horizontal stress regime. Unfortunately it was not possible to repeat the monitoring exercise in the hope of selecting a site that would later be subjected to the effects of a high horizontal stress.

### 3.4.1 Site performance summary Colliery 'A'

| | | | |
|---|---|---|---|
| Coalfield: | Vereeniging | Seam: | 2b |
| Sites: | Two | Positions: | Roadway |
| Road widths: | 5.0 m | Pillar widths: | 24 x 48 m |
| Mining height: | 3.0 m | Depth: | 70 to 80 m |



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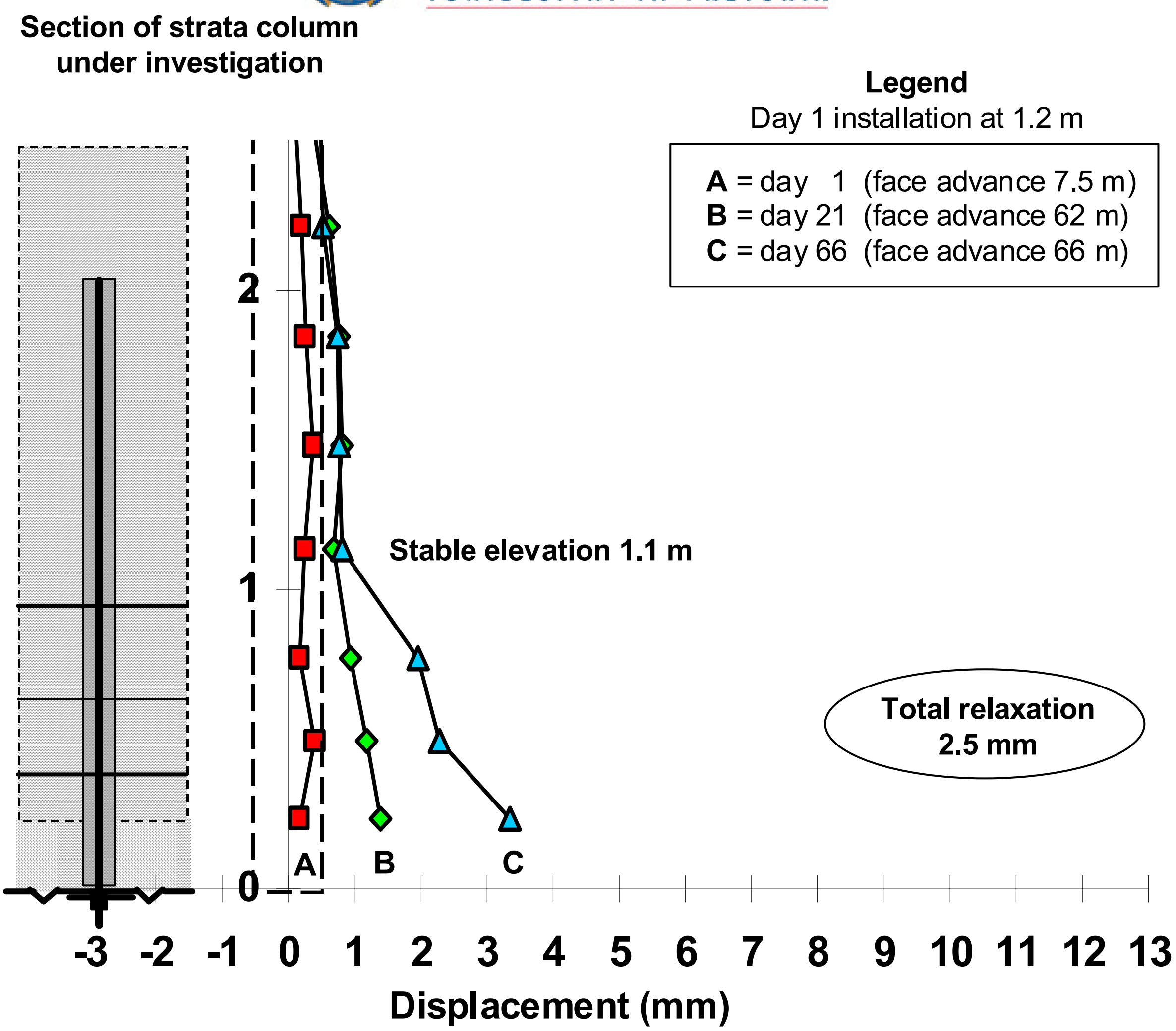
Mining method: Continuous miner, onboard roofbolters

Roof strata: Shale/mudstone

Support: 2.1 m x 21 mm AX bar, full column resin in 25 mm hole
Five to six bolts per row 1.0 m apart with “W” straps

Performance: Although there were indications of the presence of high horizontal stress within the section, there was no visual evidence to indicate its development at either monitoring site.

Roof separation was only measured at one site up to a maximum of 1.1 m into the shale allowing the roof skin a total relaxation of 2.5 mm after a face advance of 66 m over a 66 day period.




**Notes**

**Coalfield :** Vereeniging **Seam** 2b **Position** Roadway

**Roof :** Shale

**Support :** 2.1 m AX bar 21 mm diameter with full column resin in a 25 mm diameter hole. Five to six bolts per row 1.0 m apart with 'W' straps.

**Layout : Depth** 70 to 80 m **Bord** 5.0 m **Pillar** 24 x 48 m **Mining height** 3.0 m

**Mining :** Continuous miner with an onboard roof bolter.

Although there were indications of the presence of high horizontal stress within the section there was no visual evidence to indicate its development at this particular monitoring site.

***Figure 3-2 Colliery 'A' site 1 (bord)***

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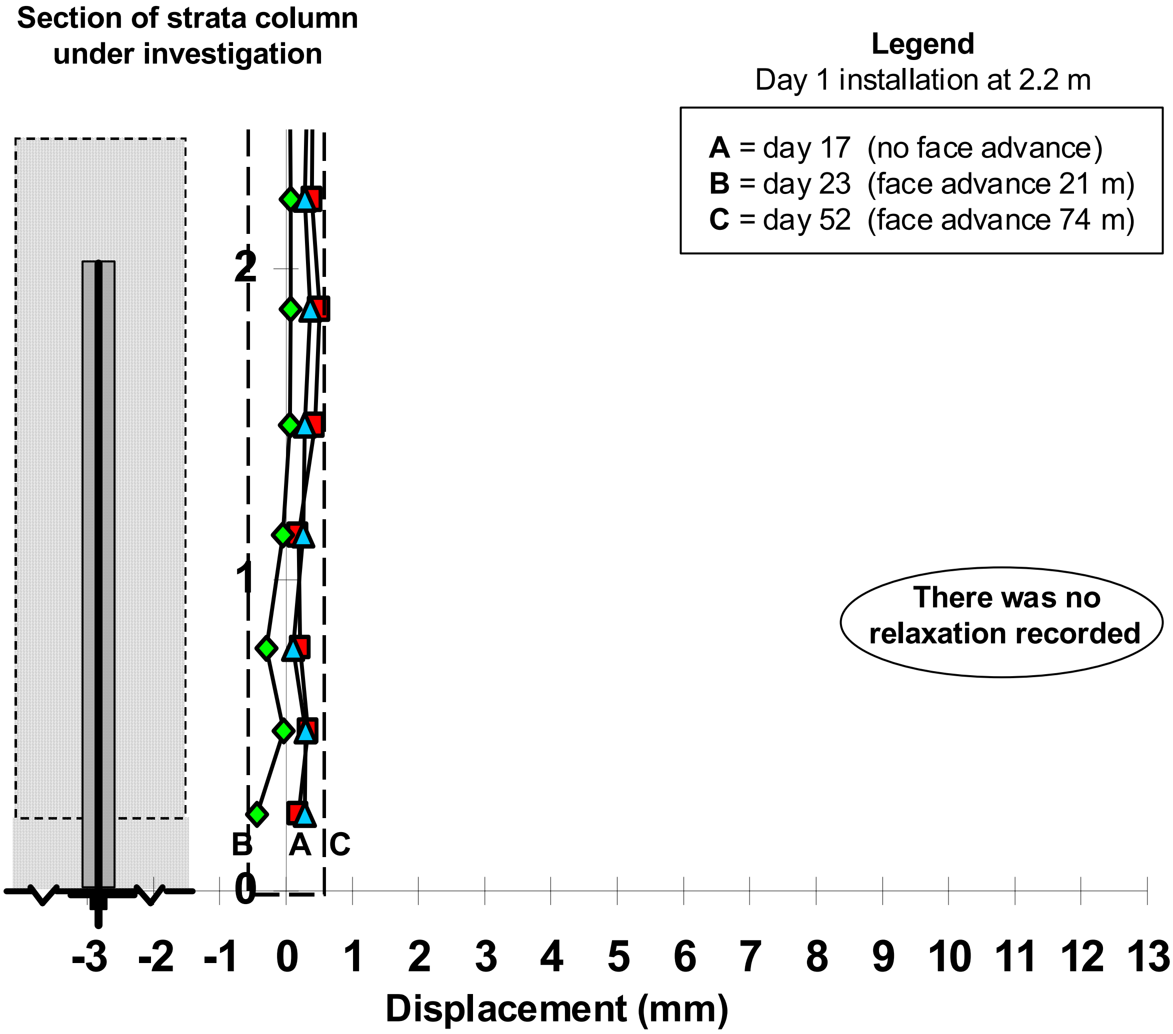



**Notes**

**Coalfield :** Vereeniging **Seam** 2b **Position** Roadway

**Roof :** Shale

**Support :** 2.1 m AX bar 21 mm diameter with full column resin in a 25 mm diameter hole. Five to six bolts per row 1.0 m apart with 'W' straps.

**Layout : Depth** 70 to 80 m **Bord** 5.0 m **Pillar** 24 x 48 m **Mining height** 3.0 m

**Mining :** Continuous miner with an onboard roof bolter.

Although there were indications of the presence of high horizontal stress within the section there was no visual evidence to indicate its development at this particular monitoring site.

*Figure 3-3 Colliery 'A' site 2 (bord)*

### 3.5 Colliery 'B'

Colliery 'B' is situated in the Witbank Coalfield and mines the No 2 Seam using conventional drill and blast mining method at an approximately 40 m depth below surface.

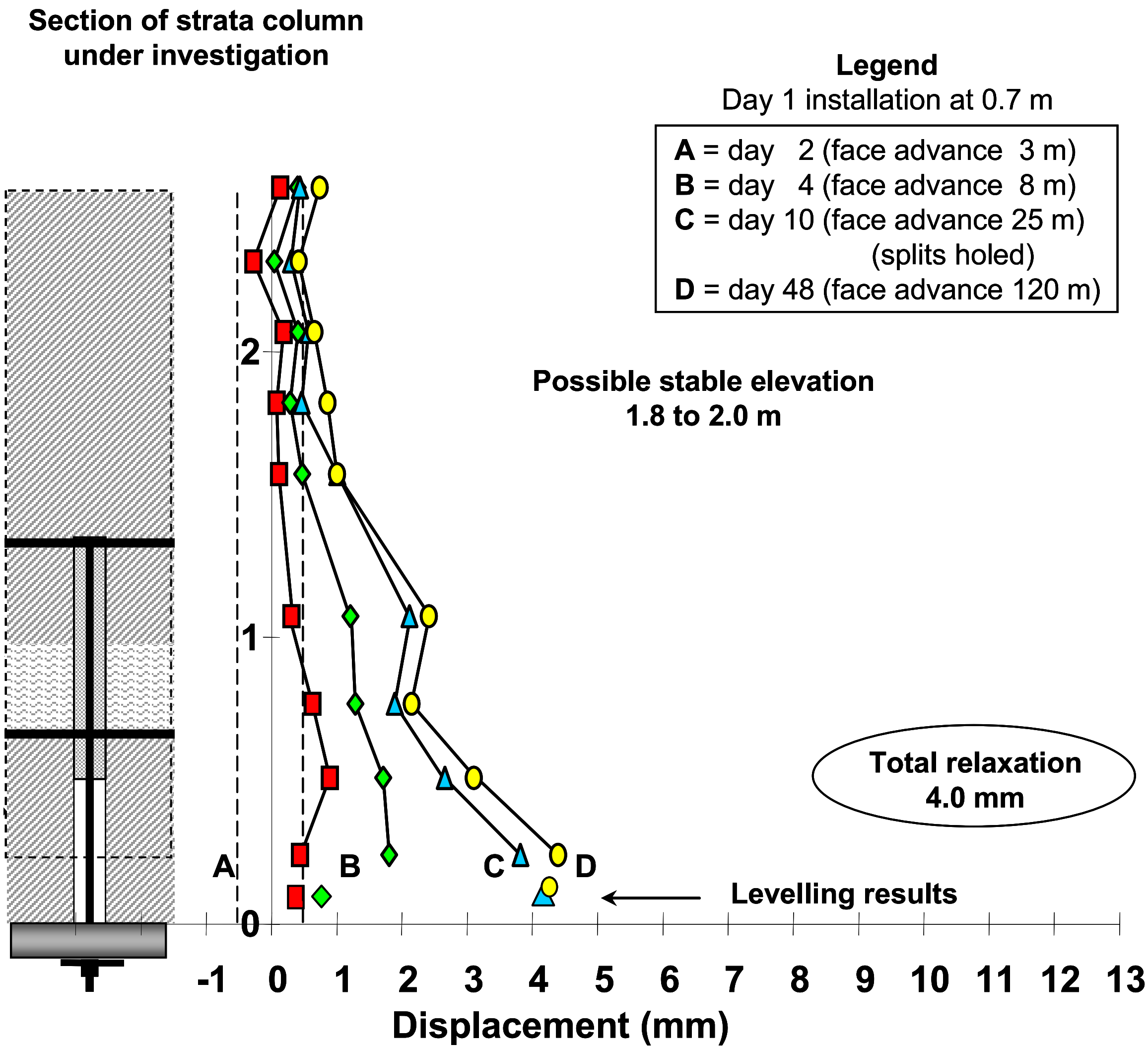
Six sites, four in roadways and two in intersections, were monitored at Colliery 'B'. The results are presented in Figure 3-4 to Figure 3-9. Note that the displacements appeared between 1.2 m to 2.2 m into the roof in Figure 3-5 are anomalies of the sonic probe extensometer, which thought to be caused by moving the anchors in the hole during pushing the probe into the hole to take readings.

In area 1, the installation and initial readings of sites 2 and 3 were taken on the same day as reading B (day four) at site 1. Similarly, in area 2 the installation and initial readings of sites 2 and 3 were taken on the same day as reading B (day six) at site 1. The monitoring in area 2 was carried out six months after the monitoring at area 1.

At three of the six monitoring sites, backup levelling was installed and monitored in conjunction with the sonic probe investigation. At all three sites, as is evident in Figure 3-4, Figure 3-6 and Figure 3-8, the levelling results agreed very closely with, and confirmed the final position and displacements of the sonic probe anchor closest to the collar of the hole at various stages during the monitoring period.

The immediate roof strata consisted of 0.5 to 1.0 m of coal, followed by a shale band approximately 0.3 m thick above which there is a further 3.0 m of coal. In the figures the 'typical' roof strata profile shows a shale band 0.3 m wide positioned at 0.7 m to 1.0 m into the roof. On a site specific basis the exact thickness and position of the shale band are not known. When comparing the six sonic probe graph results against the 'typical' strata column section, this unknown shale band elevation should be borne in mind.

Monitoring of the first three sites at area 1, an intersection and two roadways, was carried out to establish the characteristics of the particular strata combination and support performance. The opportunity to do additional monitoring at area 2 came about as the result of a dyke. In adjacent sections of the mine, separated by a dyke, there appeared to be differences in the competency of the roof although the roof strata were similar. Again an intersection and two adjacent roadways were monitored. There was a slight difference in the mining sequence at area 2 site 3 where the roadway was only advanced 3.0 m before being holed into from the opposite direction.




**Notes**

**Coalfield :** Witbank **Seam** 2 seam **Position** Intersection

**Roof :** 0.5 to 1.0 m coal, 0.3 m shale then coal to approximately 4.0m.

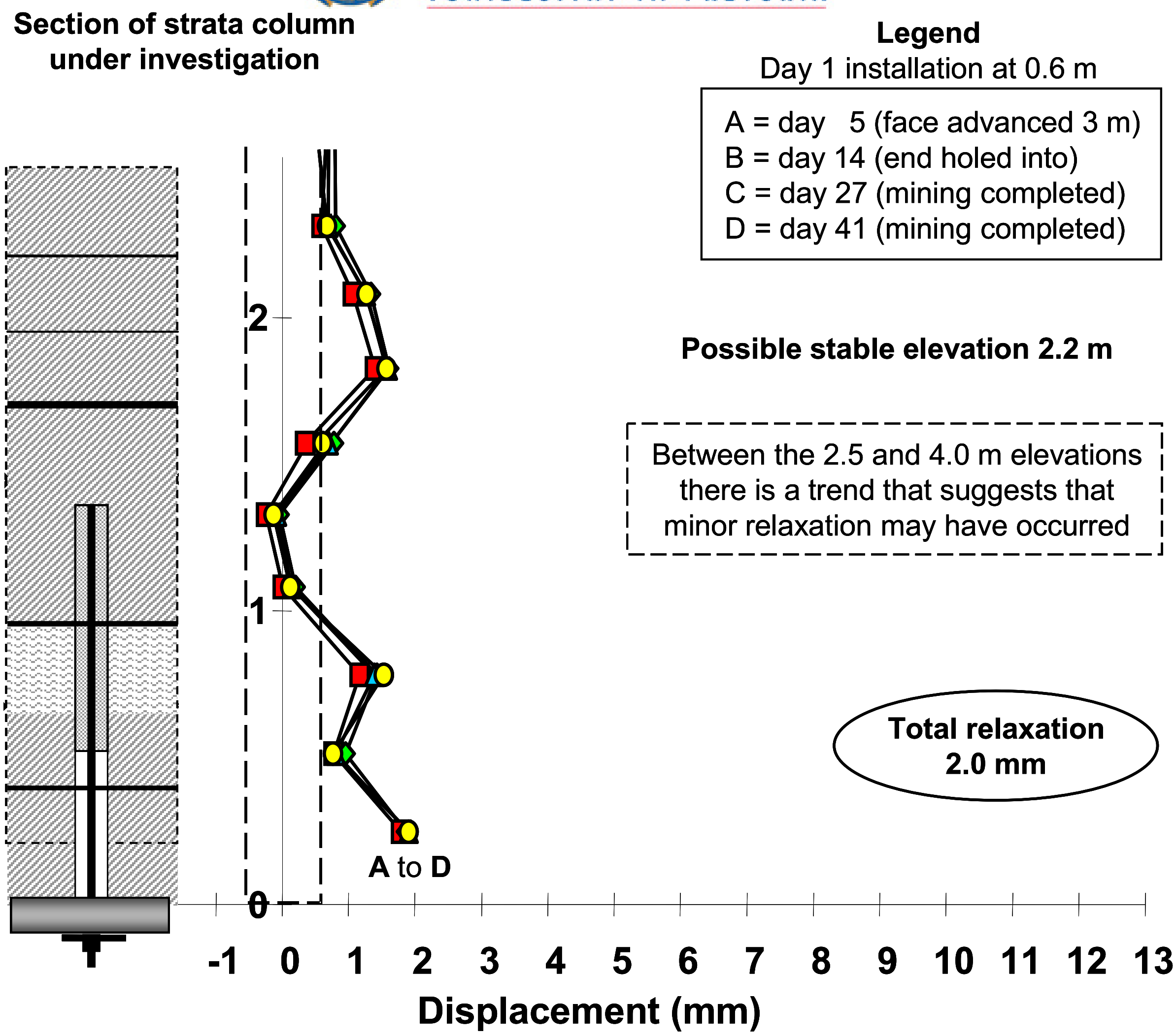
**Support :** 1.5 m x 16 mm diameter 'V' bar with partial column resin in a 24 mm diameter hole with 0.1 x 0.1 x 0.9 m headboards. Two bolts 4.0 m apart with 3.0 m between rows. Halfway between these rows is a single centre bolt in a dice five pattern.

**Layout : Depth** 40 m **Roadway** 6.0 m **Pillar** 9.0 m **Mining height** 3.0 m

**Mining :** Conventional drill and blast.

The major displacements appear to extend to just above the bolt horizon.

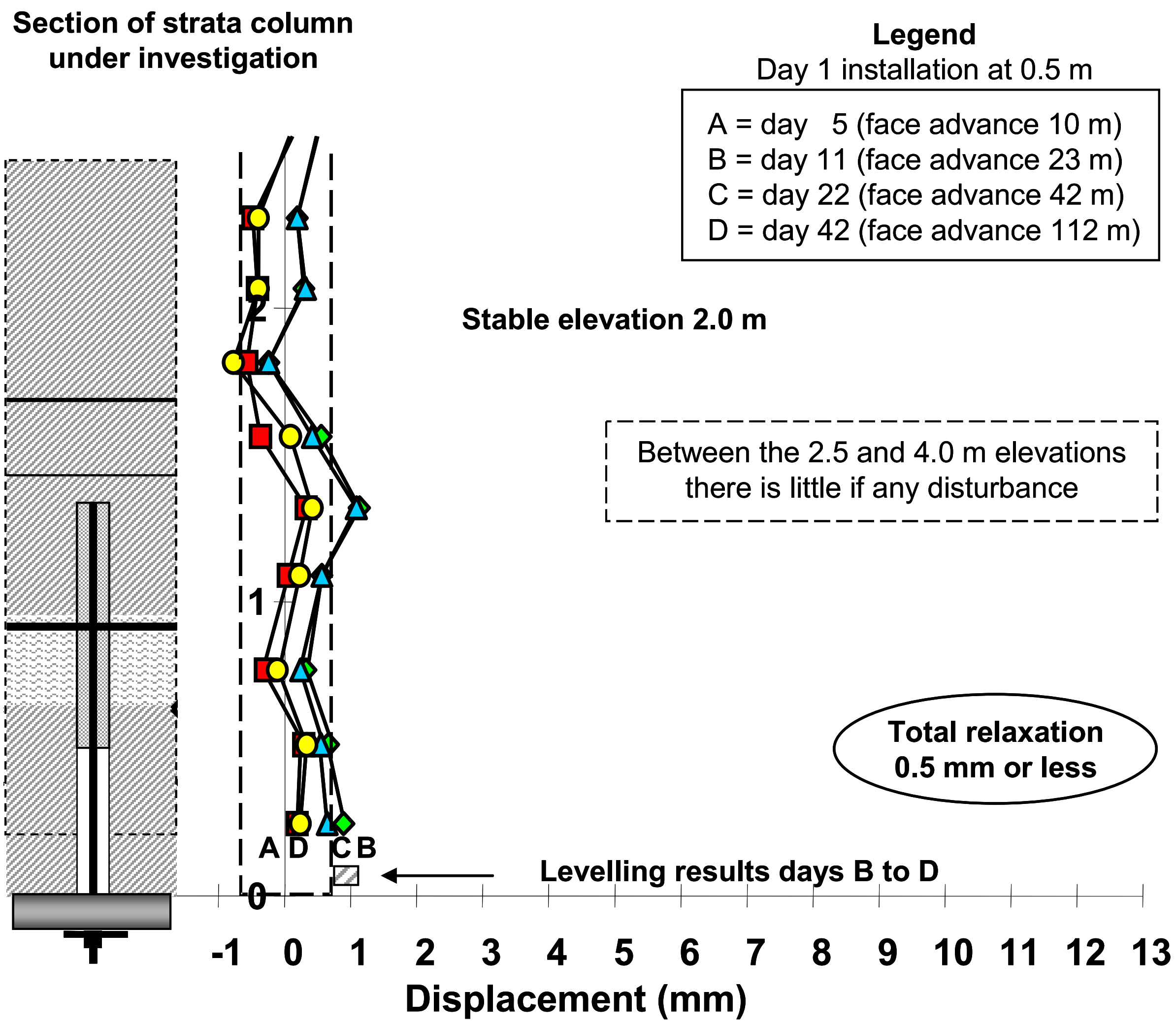
***Figure 3-4 Colliery 'B' area 1 site 1 (intersection)***




**Notes**

| |
|---|
| **Coalfield :** Witbank **Seam** 2 seam **Position** Roadway<br><br>**Roof :** 0.5 to 1.0 m coal, 0.3 m shale then coal to approximately 4.0 m.<br><br>**Support :** 1.5 m x 16 mm diameter 'V' bar with partial column resin in a 24 mm diameter hole with 0.1 x 0.1 x 0.9 m headboards. Two bolts 4.0 m apart with 3.0 m between rows. Halfway between these rows is a single centre bolt in a dice five pattern.<br><br>**Layout : Depth** 40 m **Roadway** 6.0 m **Pillar** 9.0 m **Mining height** 3.0 m<br><br>**Mining :** Conventional drill and blast.<br><br>The major displacements extend up to the 2.2 m elevation, 1.0 m above the bolt horizon. |

***Figure 3-5 Colliery 'B' area 1 site 2 (roadway)***




**Notes**

**Coalfield :** Witbank **Seam** 2 seam **Position** Roadway

**Roof :** 0.5 to 1.0 m coal, 0.3 m shale then coal to approximately 4.0 m.

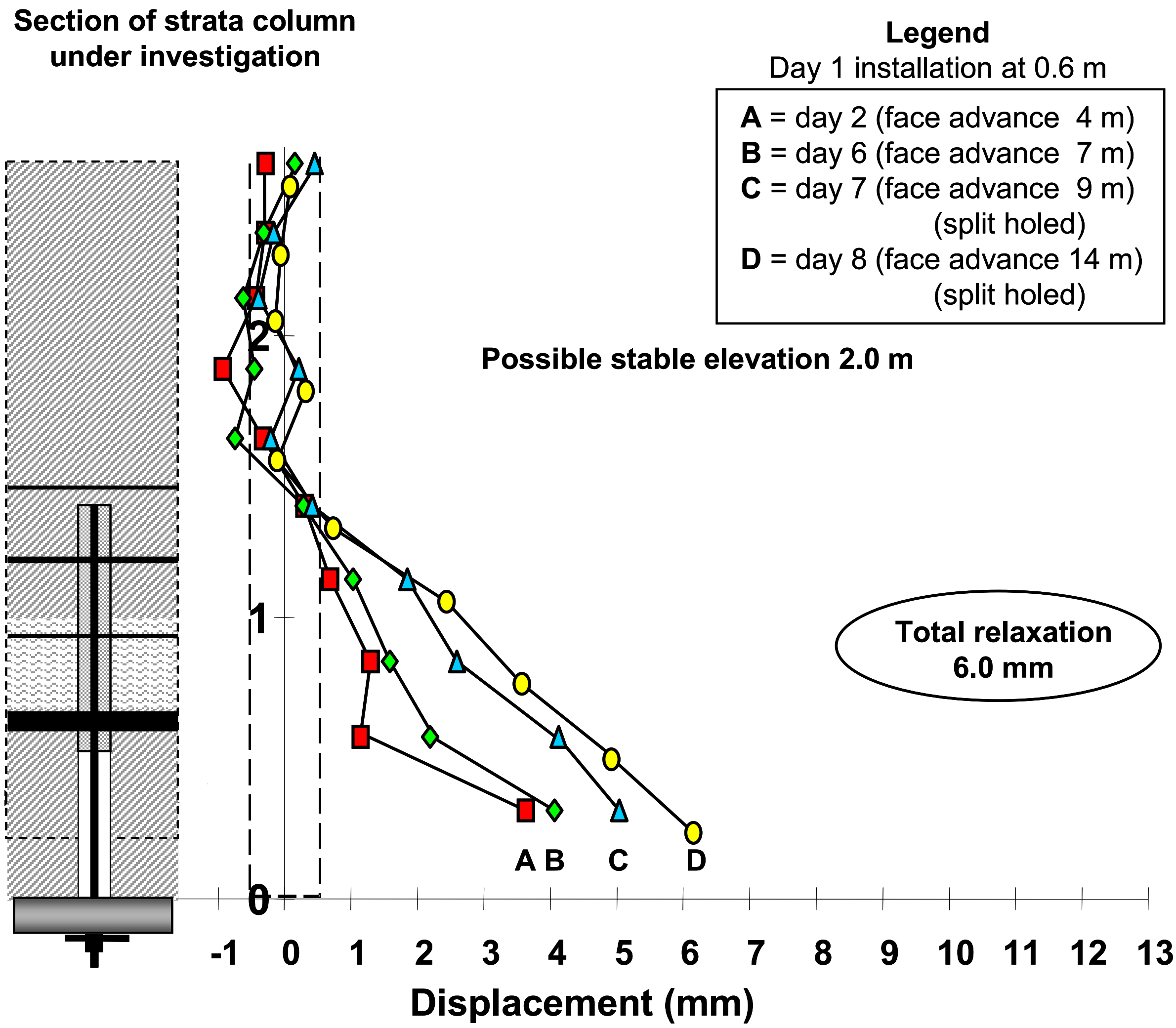
**Support :** 1.5 m x 16 mm diameter 'V' bar with partial column resin in a 24 mm diameter hole with 0.1 x 0.1 x 0.9 m headboards. Two bolts 4.0 m apart with 3.0 m between rows. Halfway between these rows is a single centre bolt in a dice five pattern.

**Layout : Depth** 40 m **Roadway** 6.0 m **Pillar** 9.0 m **Mining height** 3.0 m

**Mining :** Conventional drill and blast.

The major displacements appear to extend up to about 0.5 m above the bolt horizon.

***Figure 3-6 Colliery 'B' area 1 site 3 (roadway)***




**Notes**

**Coalfield :** Witbank **Seam** 2 seam **Position** Intersection

**Roof :** 0.5 to 1.0 m coal, 0.3 m shale then coal to approximately 4.0 m.

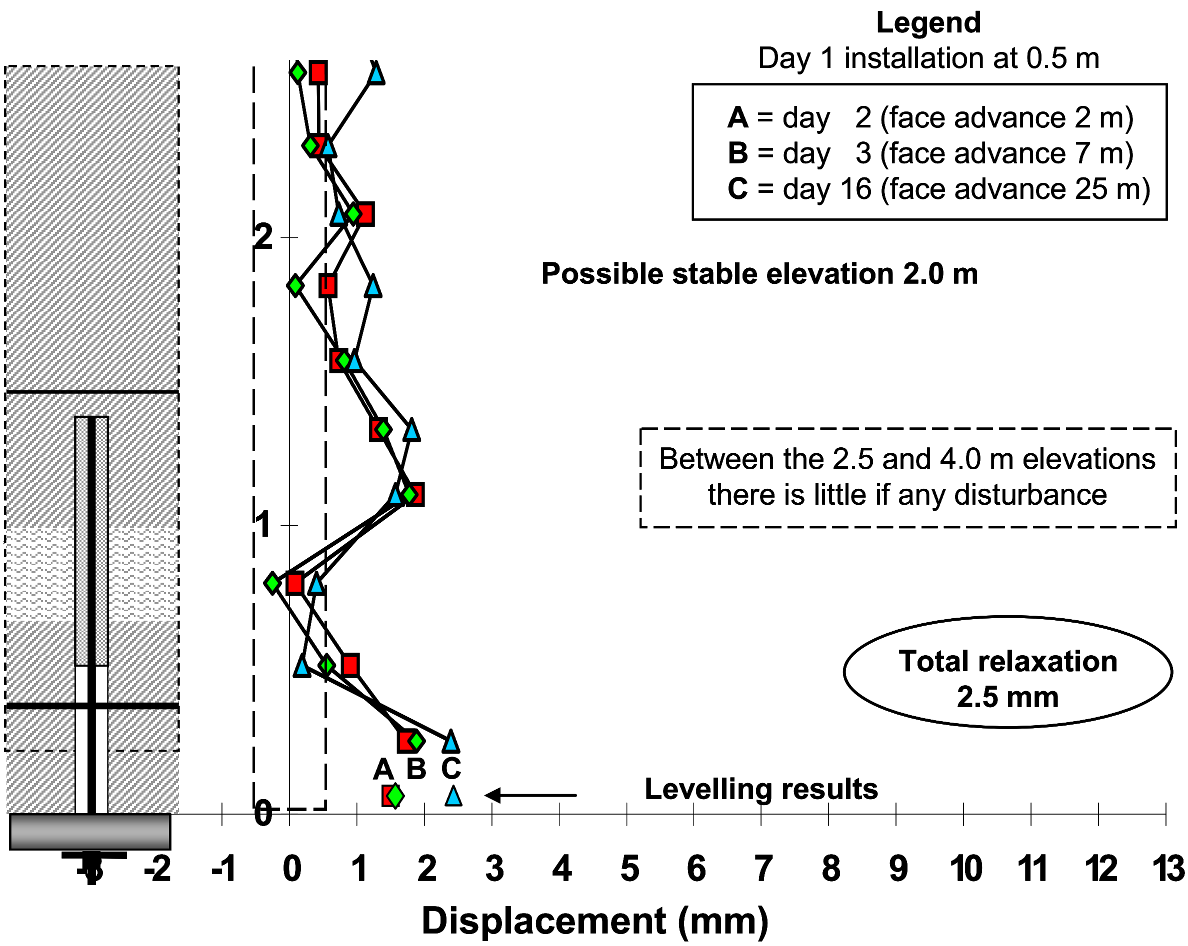
**Support :** 1.5 m x 16 mm diameter 'V' bar with partial column resin in a 24 mm diameter hole with 0.1 x 0.1 x 0.9 m headboards. Two bolts 4.0 m apart with 3.0 m between rows. Halfway between these rows is a single centre bolt in a dice five pattern.

**Layout : Depth** 40 m **Roadway** 6.0 m **Pillar** 9.0 m **Mining height** 3.0 m

**Mining :** Conventional drill and blast.

The major displacements appear to be confined to within 0.2 m above the bolt horizon.

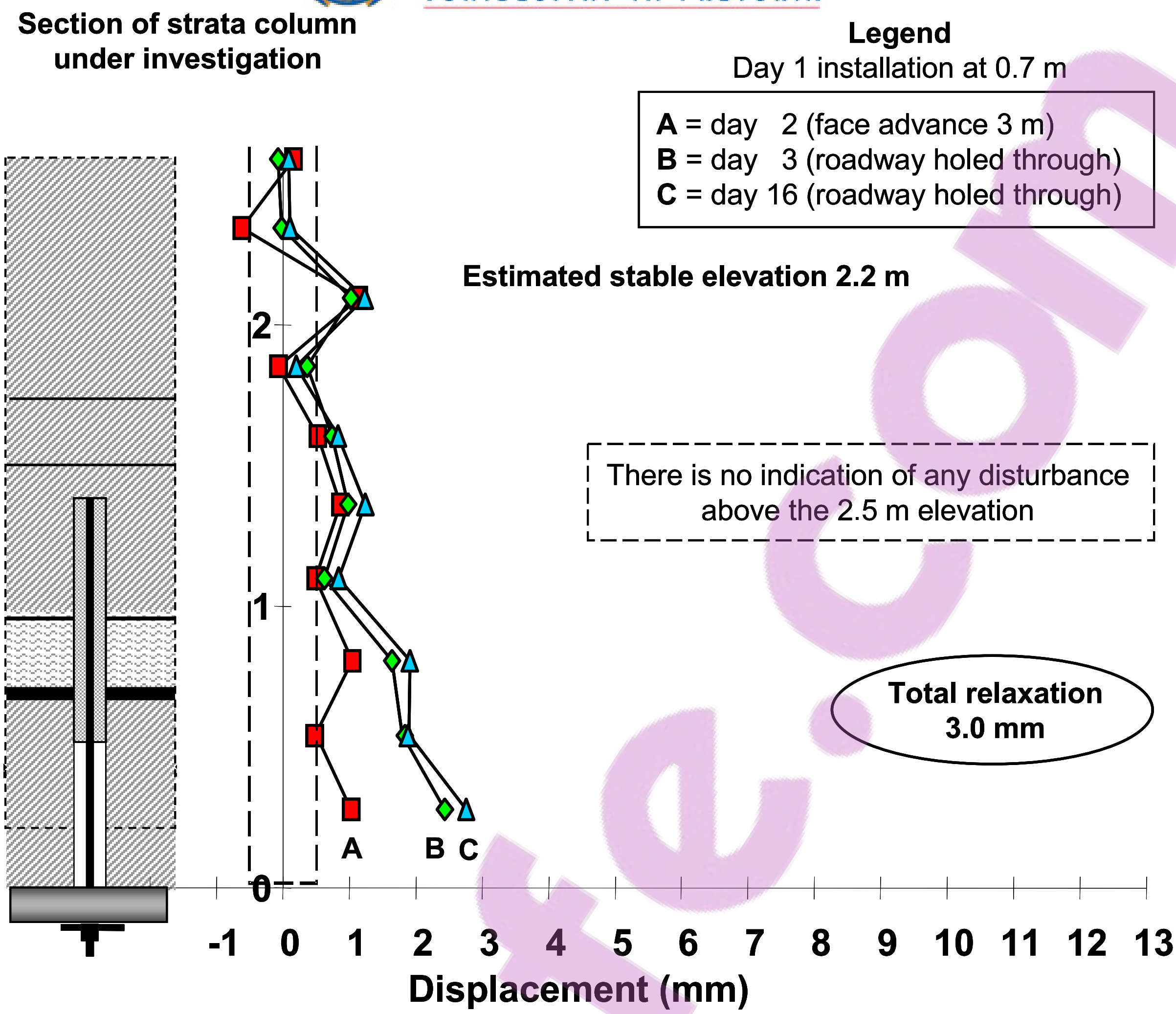
***Figure 3-7 Colliery 'B' area 2 site 1 (intersection)***




**Notes**

| |
|---|
| **Coalfield :** Witbank **Seam** 2 seam **Position** Roadway<br><br>**Roof :** 0.5 to 1.0 m coal, 0.3 m shale then coal to approximately 4.0 m.<br><br>**Support :** 1.5 m x 16 mm diameter 'V' bar with partial column resin in a 24 mm diameter hole with 0.1 x 0.1 x 0.9 m headboards. Two bolts 4.0 m apart with 3.0 m between rows. Halfway between these rows is a single centre bolt in a dice five pattern.<br><br>**Layout : Depth** 40 m **Roadway** 6.0 m **Pillar** 9.0 m **Mining height** 3.0 m<br><br>**Mining :** Conventional drill and blast.<br><br>The major displacements extend up to the 1.6 m elevation, approximately 0.2 m above the bolt horizon. A kickback of approximately 1.0 mm is situated at the 1.0 m elevation. |

***Figure 3-8      Colliery 'B' area 2 site 2 (roadway)***




**Notes**

**Coalfield :** Witbank **Seam** 2 seam **Position** Roadway

**Roof :** 0.5 to 1.0 m coal, 0.3 m shale then coal to approximately 4.0 m.

**Support :** 1.5 m x 16 mm diameter 'V' bar with partial column resin in a 24 mm diameter hole with 0.1 x 0.1 x 0.9 m headboards. Two bolts 4.0 m apart with 3.0 m between rows. Halfway between these rows is a single centre bolt in a dice five pattern.

**Layout : Depth** 40 m **Roadway** 6.0 m **Pillar** 9.0 m **Mining height** 3.0 m

**Mining :** Conventional drill and blast.

The major displacements extend up to the 1.9 or 2.2 m elevation.

*Figure 3-9 Colliery 'B' area 2 site 3 (roadway)*

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The comparative roof performance of all six sites is illustrated in Figure 3-10. From the results, the roadway in area 1 at site 2 appears to exhibit a different behaviour pattern to the other five sites with respect to the strata above the roof bolt horizon up to the 2.5 m elevation. Most of the activity in the roof strata at the other five sites is within the roof bolt horizon. The major positive opening displacements tend to be within the upper and lower limits of the 0.3 m shale band, i.e. between 0.5 and 1.3 m into the roof. This is the region where the bolts were fully resin grouted to consolidate the shale band. Although displacements are indicated in general up to 0.5 m above the bolt horizon, the magnitudes are considerably less than those recorded within the bolt horizon. The upper displacement levels at both intersections are closer to the bolt horizon than some of the roadway sites. The additional 40 per cent increase in the span across the intersection diagonals appears to have had little or no effect on crack propagation between the top of the roof bolts and the 2.5 m elevation.

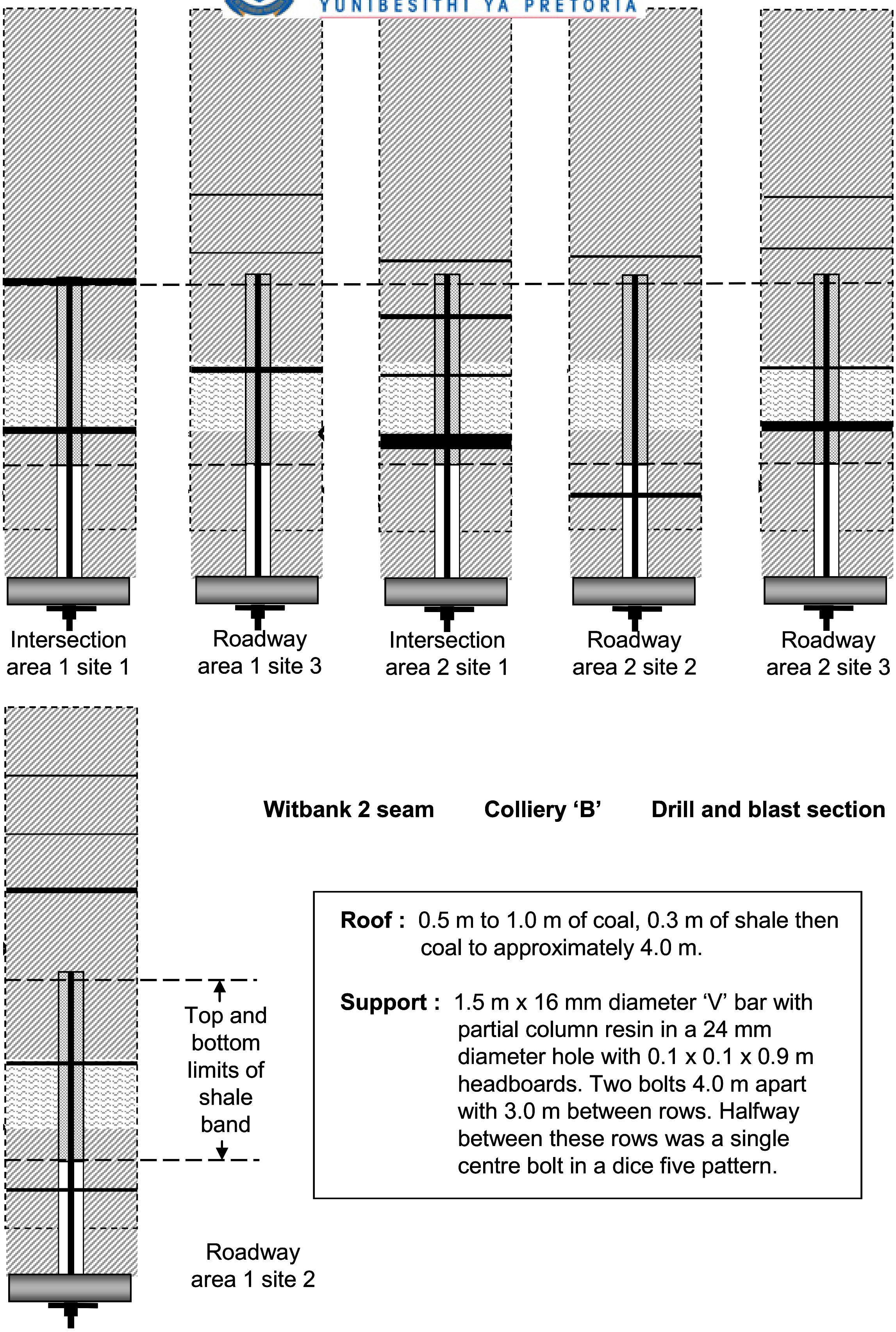


***Figure 3-10 Colliery 'B' comparative roof behaviour***

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The mining of the splits to form an intersection allowed the roof displacements to reach larger magnitudes than in the roadways. After completion of the mining cycle, the roadways and the intersections both stabilised very quickly. This is illustrated in Figure 3-11 where a comparison between the roof skin displacements, derived from the backup levelling results, at the intersection at area 1 site 1 and the roadway at area 1 site 3 are presented. The displacements of the bottom anchor near the collar of the hole in the intersection at area 1 site 1 are presented in Figure 3-12. Figure 3-13 shows the velocity profile of the same anchor. Stability was reached shortly after the splits were holed through at the 25 m face advance. The final reading was taken approximately 50 days after the initial indication that the roof had stabilised.

The overall total relaxation at the roof skin in area 2 was about 50 per cent higher than in area 1. From visual observation both roof conditions appeared to be similar with falls of ground being limited to isolated cases between the headboards.

### 3.5.1 Site performance summary Colliery 'B'

| | | | |
|---|---|---|---|
| Coalfield: | Witbank | Seam: | 2 Seam |
| Sites: | Six | Positions: | Two intersections four roadways |
| Road widths: | 6.0 m | Pillar widths: | 9.0 m |
| Mining height: | 3.0 m | Depth: | 40 m |

Mining method: Drill and blast

Roof strata: 0.5 m to 1.0 m coal, 0.3 m shale then coal to approximately 4.0 m

Support: 1.5 m x 16 mm diameter 'V' bar with partial column resin in a 24 mm diameter hole with 0.1 x 0.1 x 0.9 m headboards. Two bolts 4.0 m apart with 3.0 m between rows. Half way between these rows is a single centre bolt in a five dice pattern

Performance: Indications are that displacement occurred in the roof strata when the face was advanced with the blast, increasing the unsupported roof span up to 3.0 m. Further displacements, mainly within the roof bolt horizon, occurred quickly, within one or two blasts as the face advanced. The overall stability of the roof occurred quickly in the bords and intersections once the splits had been mined.

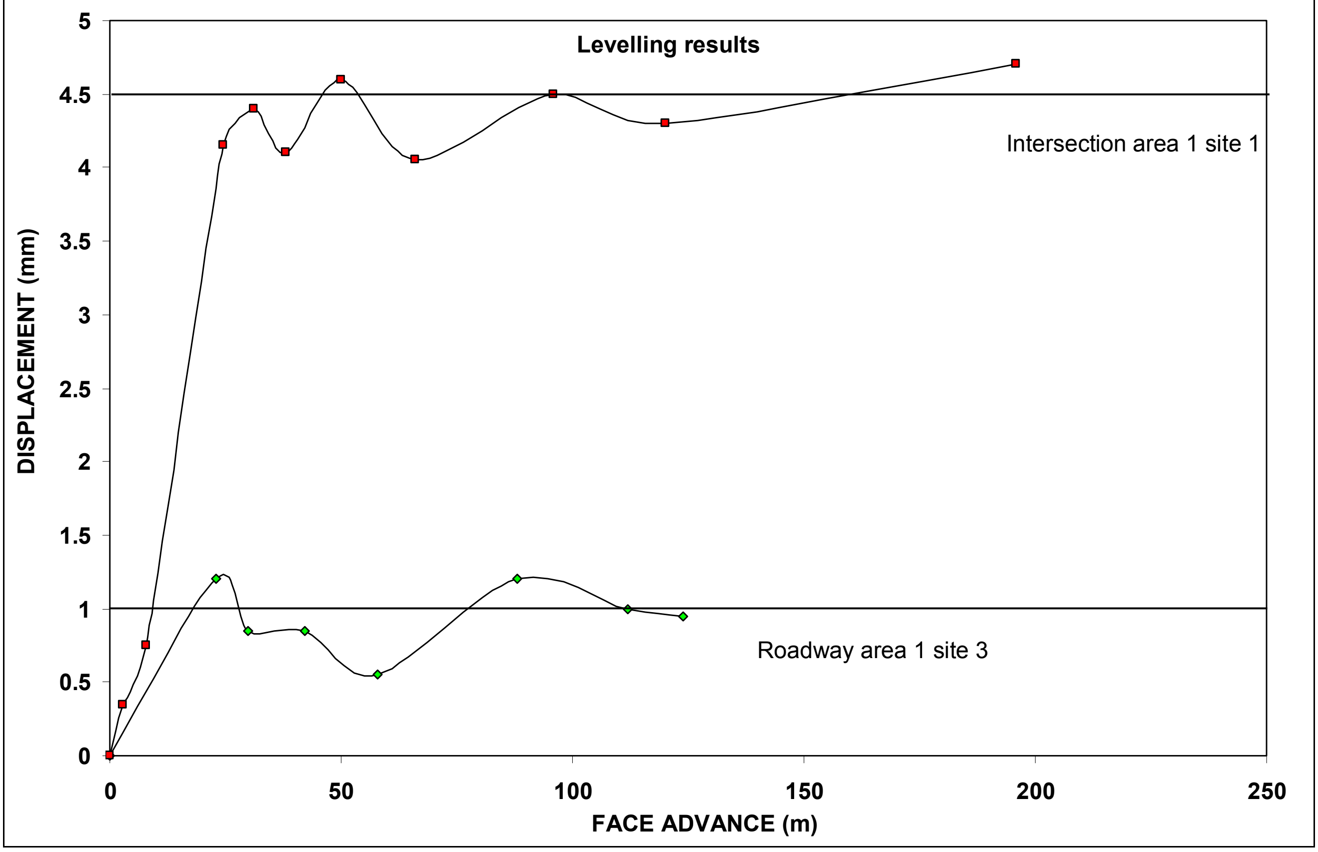


*Figure 3-11 Colliery 'B' comparison between roadway and intersection roof skin displacement*

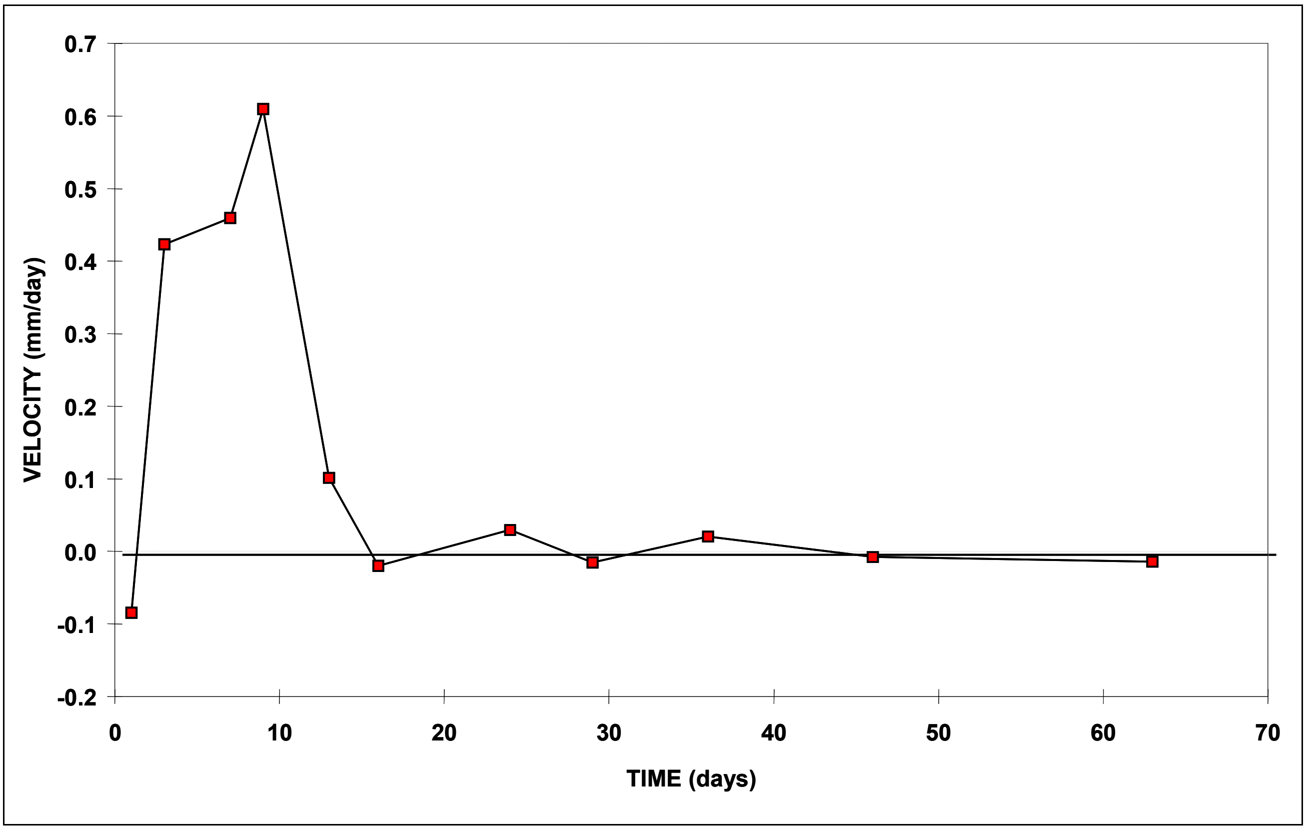


*Figure 3-12 Colliery 'B' area 1 site 1 (intersection) collar anchor displacement*

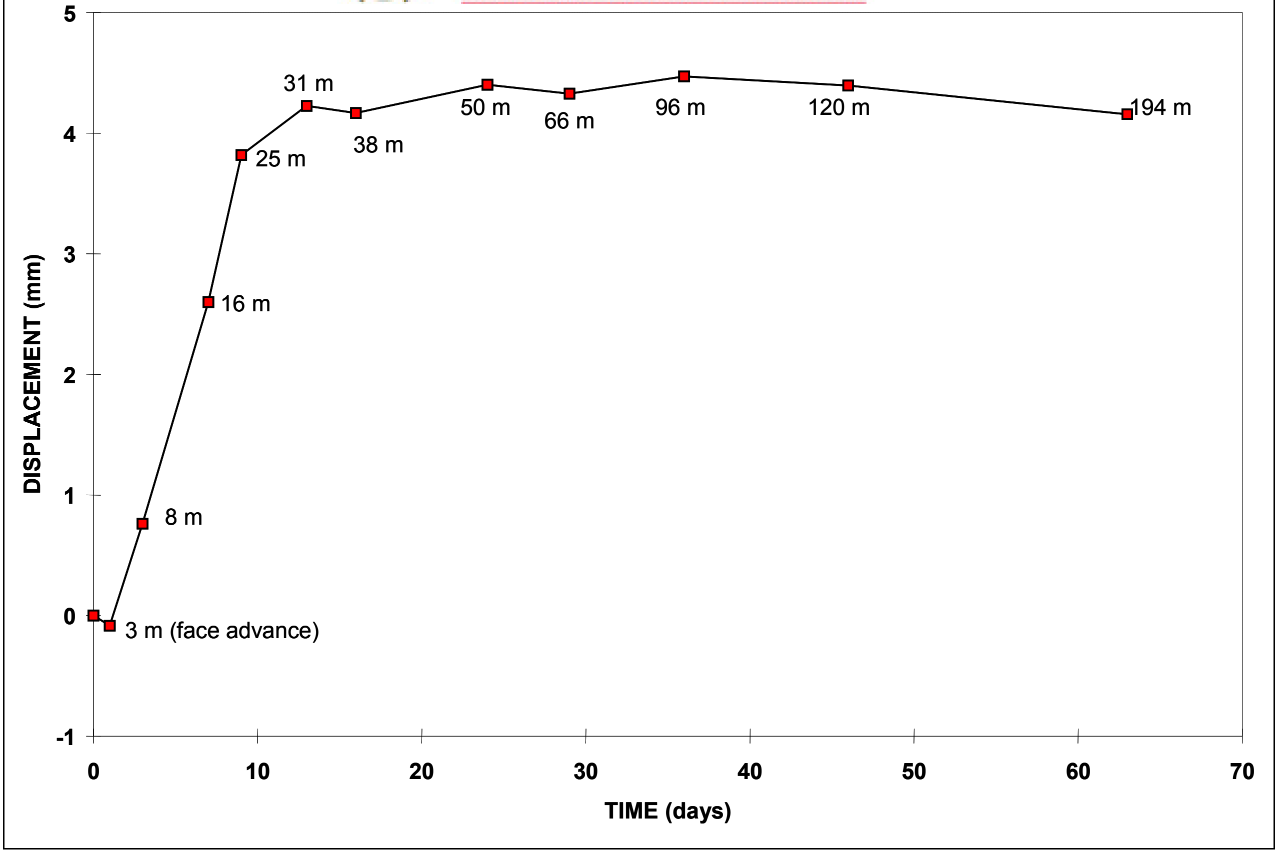


***Figure 3-13 Colliery 'B' area 1 site 1 collar anchor velocity***

## 3.6 Colliery 'C'

At Colliery 'C', which mines the No 2 Seam in the Witbank Coalfield, back up levelling results were only viable at two of the four sites as a result of blast damage to levelling installations.

Sites 1 and 2, an intersection and adjacent roadway respectively, were situated approximately 25 m away from sites 3 and 4, another intersection and roadway. The immediate roof consisted of a coal layer approximately 0.3 m thick with shale above it. The standard support was 1.8 m x 16 mm mechanical end anchored bolts, three in a row with the rows 2.0 m apart. The boltholes were drilled with electric hand held drills. The bolts were installed and tensioned by using the electric drills.

The results from Collier 'C' are presented in Figure 3-14 to Figure 3-17. The total relaxation, which occurred within or close to the roof bolt horizon, was very small. The largest displacements were recorded at the intersection in site 3, Figure 3-16, where the total relaxation was 3.5 mm. The final levelling result value was 30 per cent larger than indicated by the lowest sonic probe anchor close to the collar of the hole. This indicates the presence of an additional displacement of approximately 1.5 mm between 0.1 and 0.2 m in from the roof skin. These were the elevations of the levelling skin anchor and the bottom sonic probe anchor, respectively. At

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site 2 the levelling results and the lowest sonic probe anchor close to the collar of the hole gave near identical values.

As was the case in Colliery 'B', roof displacement in the form of open fractures or bedding planes appeared to occur very close to the face (within 0.5 m) as the blasting extended the unsupported roof span up to a maximum of approximately 3.0 m. Most of the subsequent displacements that occurred after the installation of the support and instrumentation were close to or within the roof bolt horizon, as illustrated in Figure 3-18. In general, the roof displacements appeared to have stabilised when the face had advanced by the bord width i.e. 6.0 m.

### 3.6.1 Site performance summary Colliery 'C'

| | | | |
|---|---|---|---|
| Coalfield: | Witbank | Seam: | 2 Seam |
| Sites: | Four | Positions: | Two intersections  two roadways |
| Road widths: | 6.0 m | Pillar widths: | 9.0 m |
| Mining height: | 2.2 m | Depth: | 50 to 60 m |

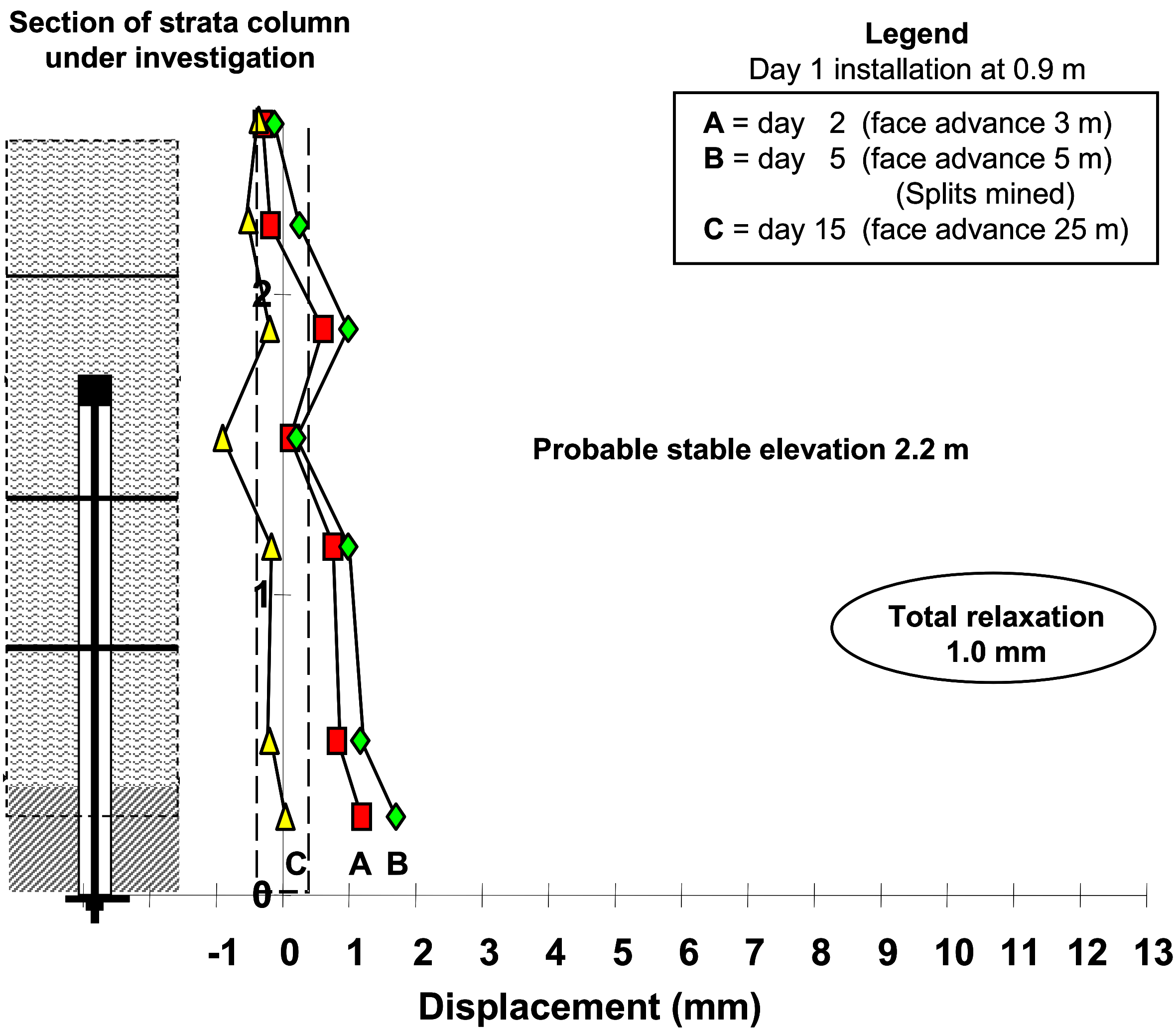
Mining method: Drill and blast

Roof strata: Approximately 0.3 m of coal with shale above it

Support: 1.8 m x 16 mm diameter mechanical end anchored bolts.
Three bolts per row with rows 2.0 m apart.

Performance: Indications are that displacement occurred in the roof strata when the face was advanced with the blast, increasing the unsupported roof span by up to 3.0 m. This resulted in open parting planes and fractures being present as close as 0.5 m from the face prior to the installation of the support and instrumentation.

Although there were isolated cases of openings restricted to approximately 0.3 m and 1.0 m into the roof, three of the four sites indicate the presence of an opening around the 2.0 m elevation just above the bolt horizon. Further small displacements, mainly within the roof bolt horizon, occurred during the first one or two blasts as the face advanced. However, stability of the roof also occurred quickly. In the intersections this was attained once the splits had been mined.




**Notes**

**Coalfield :** Witbank **Seam** 2 seam **Position** Intersection

**Roof :** Approximately 0.3 m of coal with shale above it.

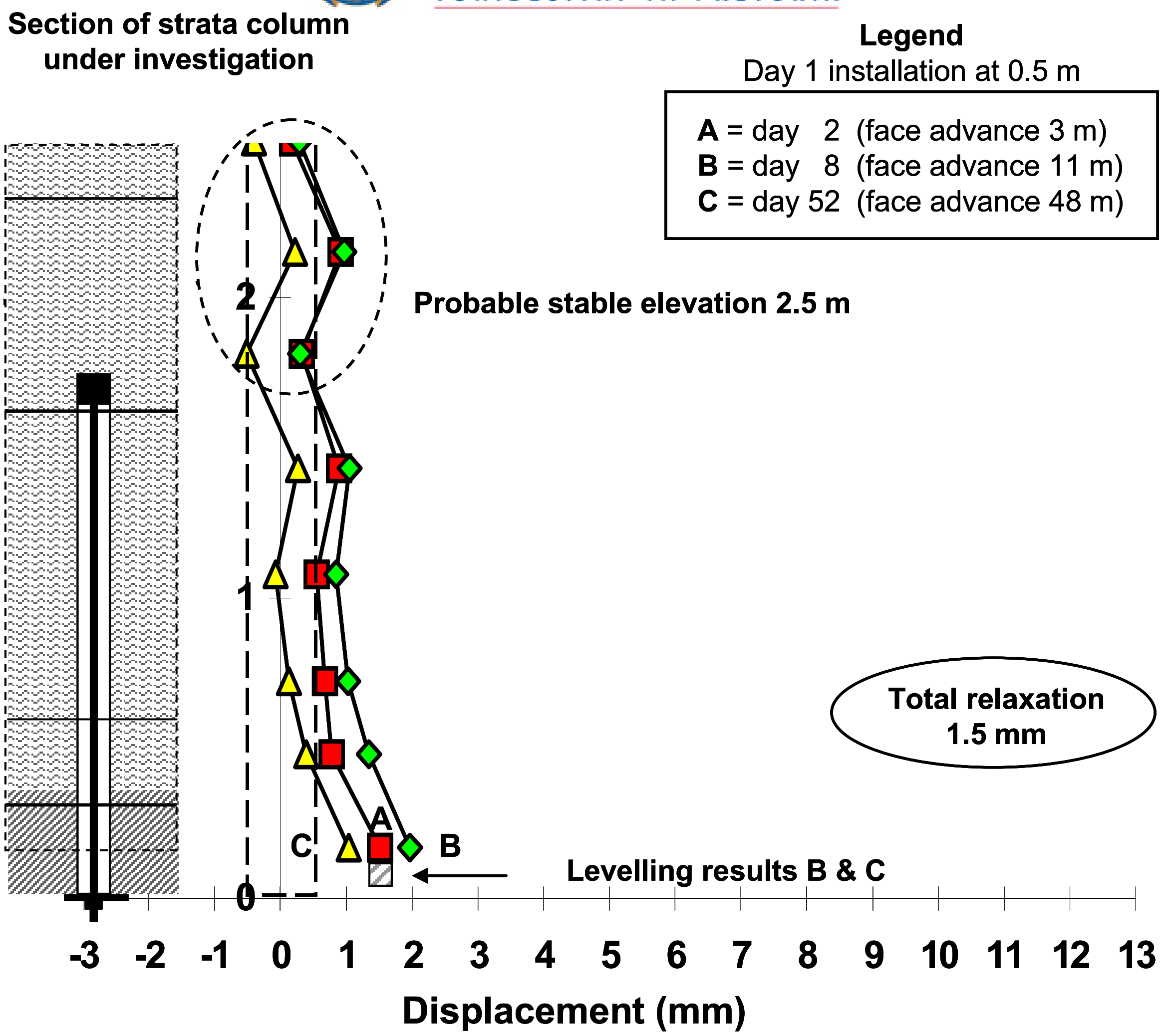
**Support :** 1.8 m x 16 mm mechanical end anchored bolts 3 in a row with rows 2.0 m apart. Bolt holes drilled with electric hand held drills.

**Layout : Depth** 50 to 60 m **Roadway** 6.0 m **Pillar** 9.0 m

**Mining :** Conventional drill and blast **Mining height** 2.2 m

Displacements appeared to have occurred up to 0.4 m above the bolt horizon.

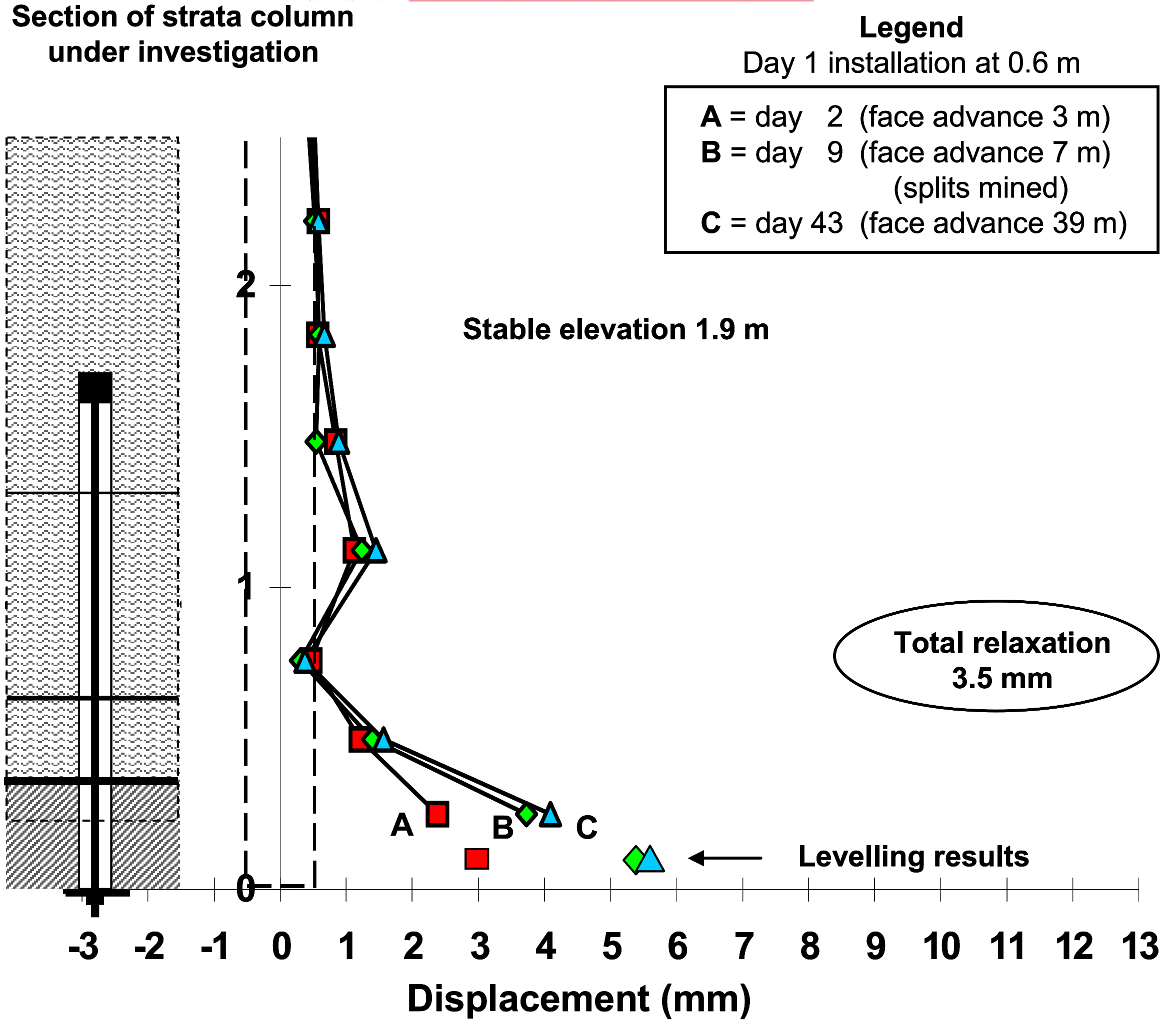
***Figure 3-14 Colliery 'C' site 1 (intersection)***




**Notes**

| |
|---|
| **Coalfield :** Witbank **Seam** 2 seam **Position** Roadway<br><br>**Roof :** Approximately 0.3 m of coal with shale above it.<br><br>**Support :** 1.8 m x 16 mm mechanical end anchored bolts 3 in a row with rows 2.0 m apart. Bolt holes drilled with electric hand held drills.<br><br>**Layout : Depth** 50 to 60 m **Roadway** 6.0 m **Pillar** 9.0 m<br><br>**Mining :** Conventional drill and blast **Mining height** 2.2 m<br><br>Displacements appear to have occurred as high as 0.7 m above the bolt horizon with a single kickback close to the 2.0 m elevation. |

*Figure 3-15 Colliery ‘C’ site 2 (roadway)*




**Notes**

**Coalfield :**Witbank **Seam** 2 seam **Position** Intersection

**Roof :** Approximately 0.3 m of coal with shale above it.

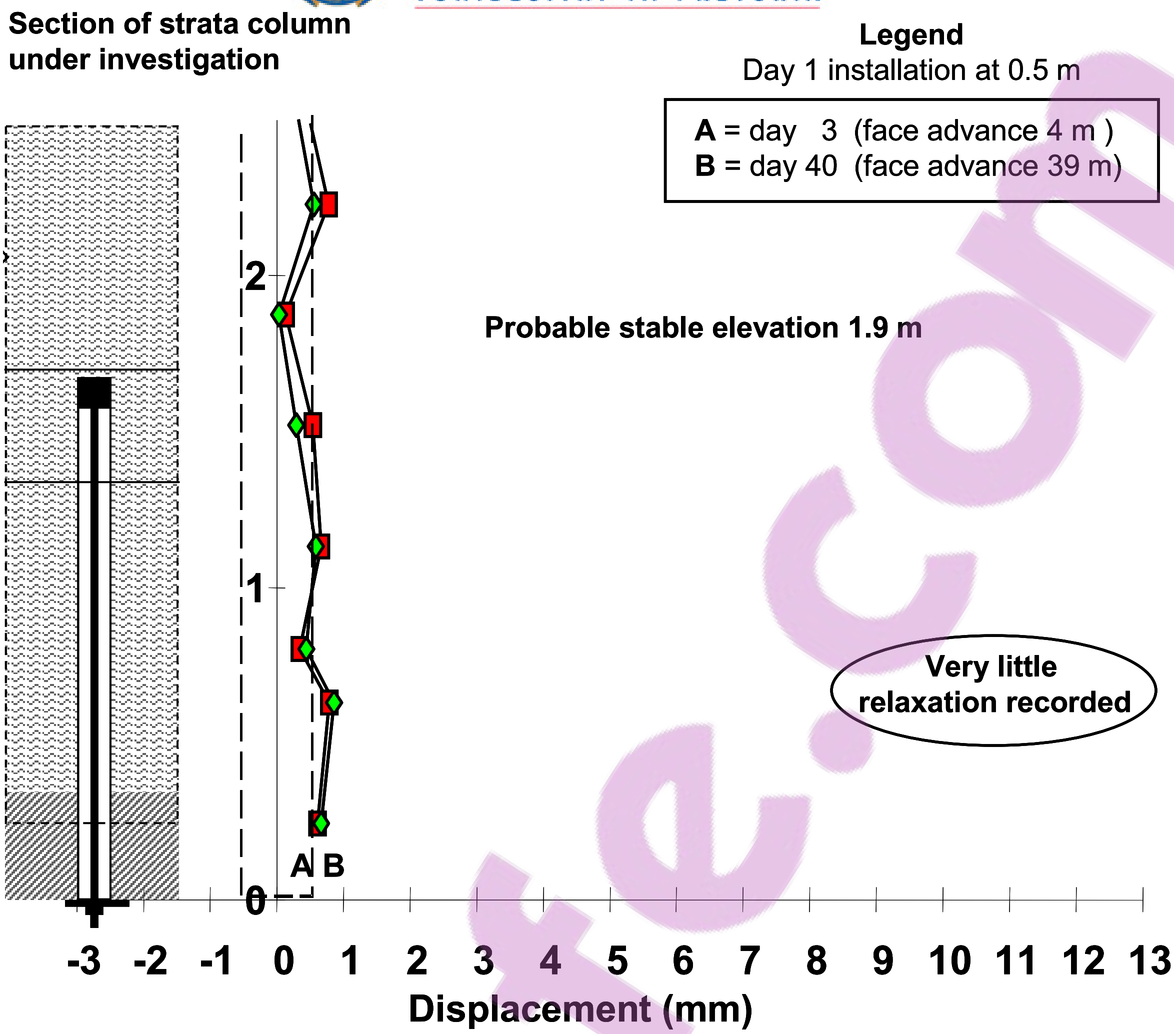
**Support :** 1.8 m x 16 mm mechanical end anchored bolts 3 in a row with rows 2.0 m apart. Bolt holes drilled with electric hand held drills.

**Layout : Depth** 50 to 60 m **Roadway** 6.0 m **Pillar** 9.0 m

**Mining :** Conventional drill and blast **Mining height** 2.2 m

All the displacements appear to have taken place within or close to the bolt horizon. There was a kickback at approximately the 1.0 m elevation. Positive opening displacements of about 4.0 mm in total occurred within the initial 0.8 m of roof strata.

*Figure 3-16 Colliery 'C' site 3 (intersection)*




**Notes**

**Coalfield :** Witbank **Seam** 2 seam **Position** Roadway

**Roof :** Approximately 0.3 m of coal with shale above it.

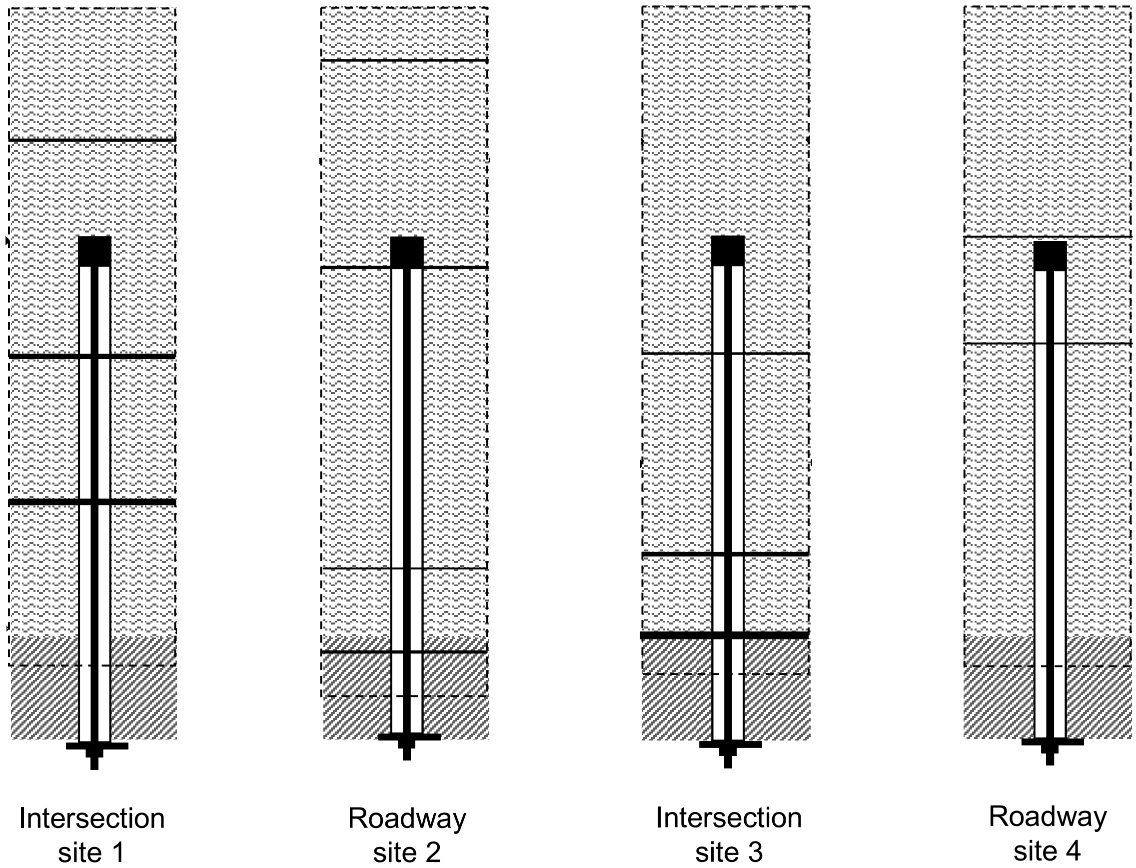
**Support :** 1.8 m x 16 mm mechanical end anchored bolts 3 in a row with rows 2.0 m apart. Bolt holes drilled with electric hand held drills.

**Layout : Depth** 50 to 60 m **Roadway** 6.0 m **Pillar** 9.0 m

**Mining :** Conventional drill and blast **Mining height** 2.2 m

The overall displacements recorded, although very small, extended to just above the bolt horizon.

*Figure 3-17 Colliery 'C' site 4 (roadway)*




**Roof :** Approximately 0.3 m of coal with shale above it.

**Support :** 1.8 m x 16 mm mechanical end anchored bolts three in a row with rows 2.0 m apart. Bolt holes drilled with electric hand held face drills. Bolts tensioned using the electric drill.

***Figure 3-18 Colliery 'C' comparative roof behaviour***

## 3.7 Colliery 'D'

Monitoring was carried out at three different locations at Colliery 'D'. A total of 12 sites were monitored covering four support combinations and two mining methods. The roof consisted of laminated sandstone and shale with variable bedding thicknesses. The support pattern of four bolts per row with rows 1.5 m apart remained the same at all the sites. Backup levelling was carried out at eight of the sites.

In the first area, two intersections and two roadways were monitored. The support method used was 1.5 m long 15 mm spiral bars. These were installed in 22 mm diameter holes using three 19 x 380 mm resin cartridges giving a full column resin bond. The mining method used was conventional drill and blast. As is usual practice, each blast advanced the full face width of 6.0 m by approximately 2.0 m. This resulted in unsupported maximum exposed roof distances, from the last row of support up to the face, of approximately 3.5 m. The monitoring hole was always within 1.0 m of the nearest roof bolt.

The individual monitoring results from site 1 are presented in Figure 3-19, Figure 3-20, Figure 3-21 and Figure 3-22. The levelling results at all four sites were similar to, but generally had slightly larger values than those indicated by the lowest sonic probe anchor.

The intersection at site 1 (Figure 3-19) was the only site that appeared to have experienced displacements above the bolt horizon. It is difficult to determine if there was any displacement in the 0.5 m above the bolt horizon in the other intersection at site 3 (Figure 3-21) due to what appear to be anomalous readings. The average total relaxation experienced at the intersections was 4.5 mm whereas that in the roadways was less than 1.0 mm.

In the roadway at site 4 there were indications of very small displacements. These were however all less than the accepted accuracy band of the sonic probe extensometer and were therefore not transferred onto the strata column. The smallest face advance that took place before the second set or readings were taken was 4.0 m at site 4. The displacements that were recorded had virtually all taken place by the time of the second visit.

All four sites have been grouped together in Figure 3-23.

The levelling results of the roof skin behaviour relative to the 1.8 m datum, for all four sites, have been plotted and are presented in Figure 3-24. To compare roadway roof behaviour it is easy and probably more accurate to use face advance as opposed to time as one of the axes. The complex nature of "face advance" during the development of an intersection introduces

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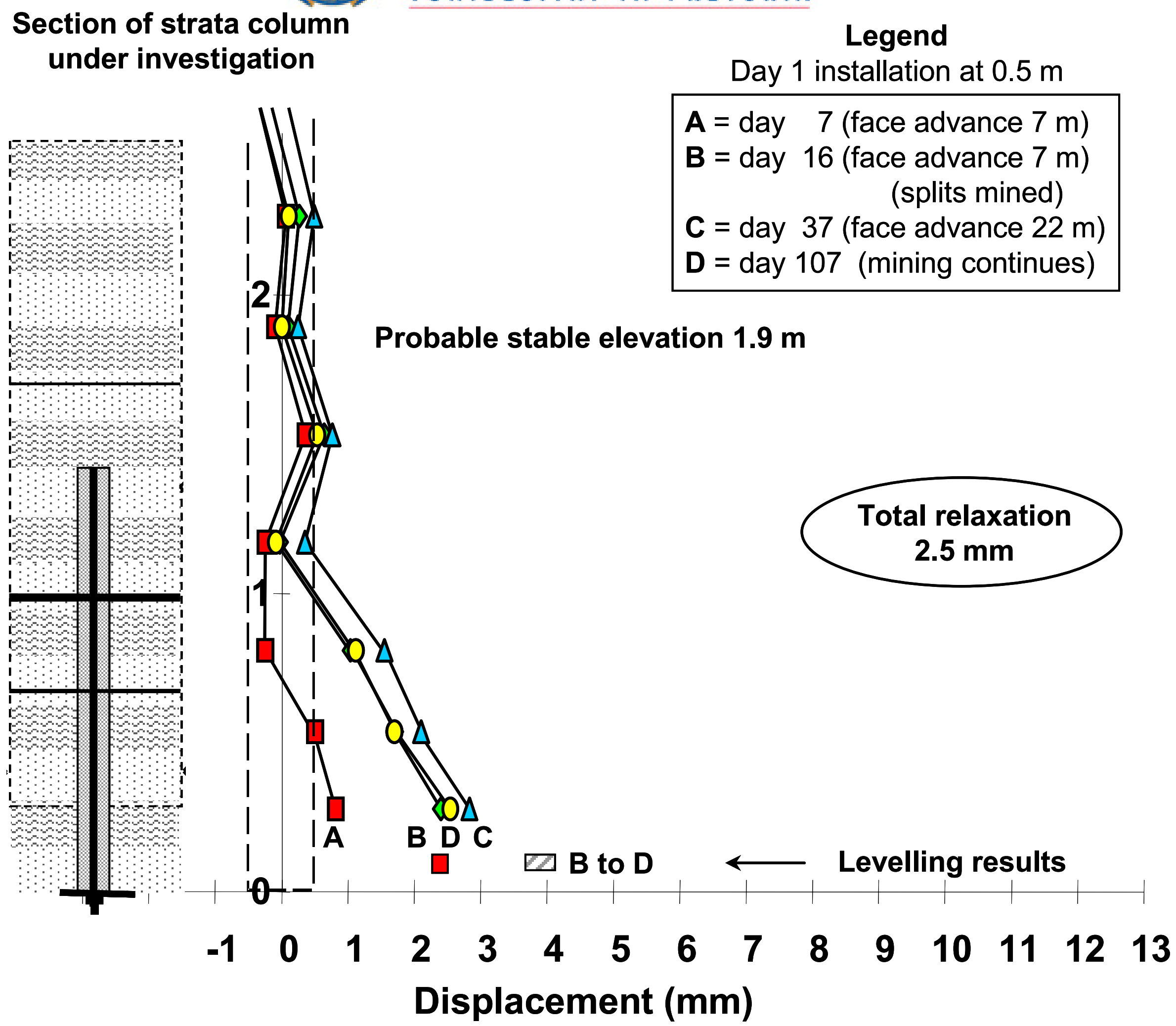
complications, particularly when comparing the development of an intersection to a roadway, as well as one intersection to another. Although not ideal, displacement with time is considered to be the better option in this case.

At site 1 and two, between days 20 and 40, readings which were larger magnitudes than the accepted accuracy of the levelling system of 0.5 mm were recorded, Figure 3-24. In both intersections, at sites 1 and 3, the step like behaviour of the displacements can be seen between days 8 and 13 during the time when the splits were being developed.

At the second area monitored at Colliery ‘D’, both the support system and mining method were different. The support pattern and tendon type remained the same as in area 1, however, the resin was reduced to two 19 x 380 mm cartridges. This resulted in a partial column resin bond length of approximately 1.04 m. This left the initial 0.4 m of roof bolt in from the roof skin resin free. Mining was carried out using a continuous miner.

Four of the five sites were intersections. The intention was to install 0.1 X 0.1 x 0.9 m wooden headboards (areal coverage used with roof bolts in stead of metal washer) in two intersections to determine what affect this had on the roof behaviour. Unfortunately, due to a shortage of headboards at the time, they were only installed at site 1. Adding the headboards appeared to reduce the effective roof bolted horizon from 1.45 m to 1.35 m. Backup levelling to confirm this was installed at four of the five sites.




**Notes**

**Coalfield :**Witbank **Seam** 2 seam **Position** Intersection

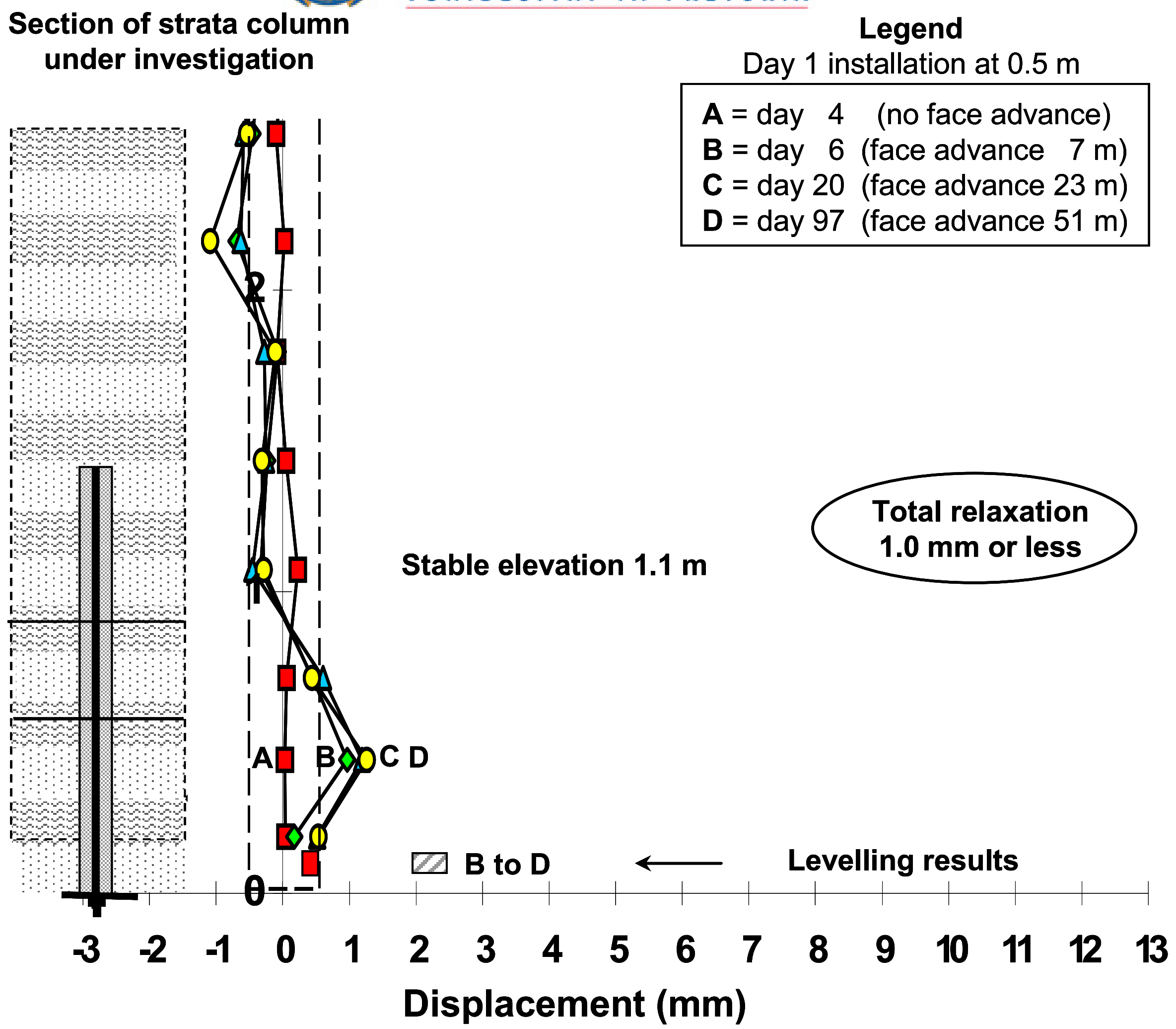
**Roof :** Laminated sandstone & shale with highly variable bedding thicknesses

**Support :** 1.5 m x 15 mm spiral bars with full column resin in 22 mm diameter holes. Four bolts per row with rows 1.5 m apart.

**Layout : Depth** 55 to 60 m **Roadway** 6.0 m **Pillar** 6.0 m

**Mining :** Conventional drill and blast **Mining height** 2.1 m

*Figure 3-19 Colliery 'D' area 1 site 1 (intersection)*




**Notes**

**Coalfield :**Witbank **Seam** 2 seam **Position** Roadway

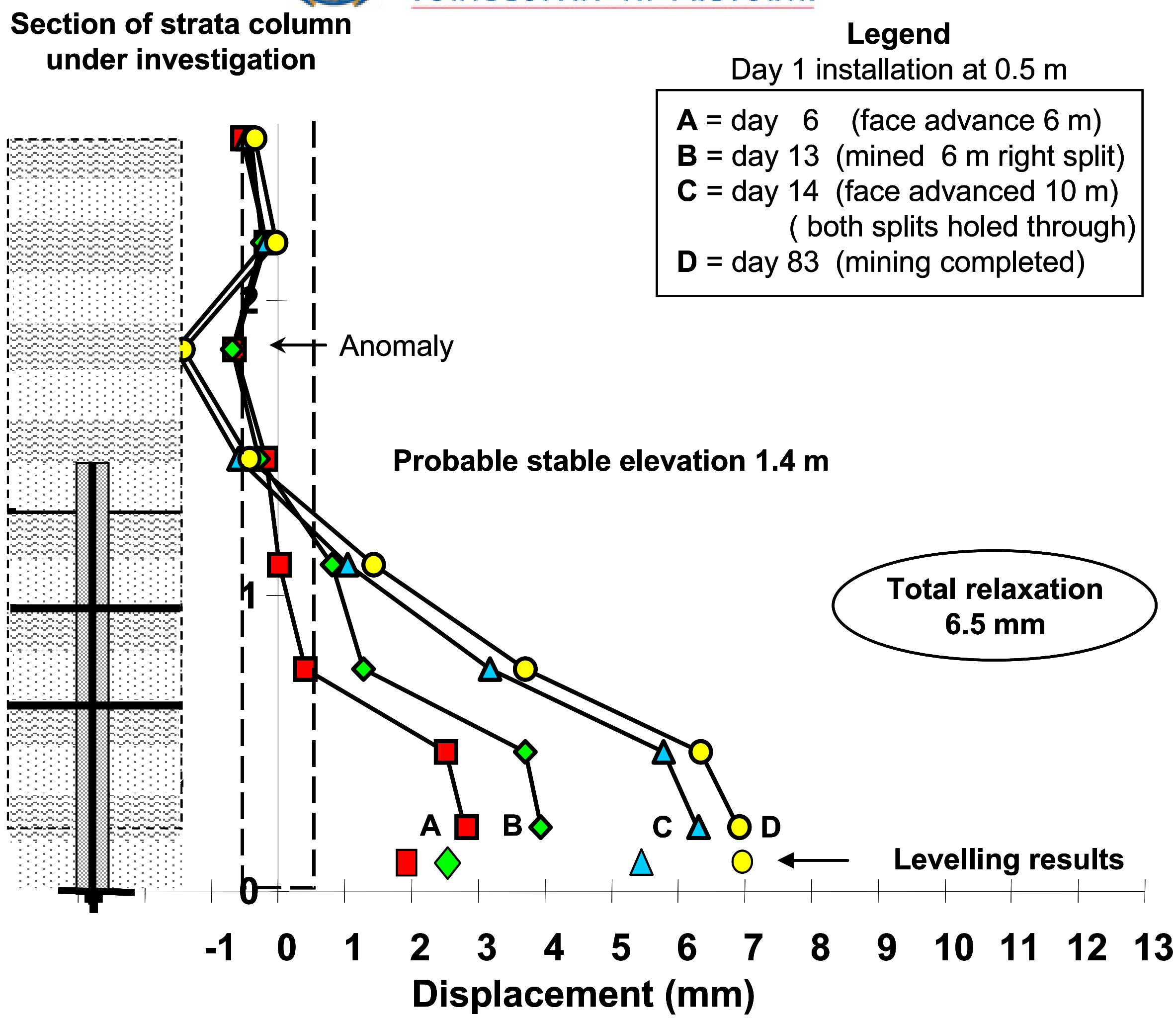
**Roof :** Laminated sandstone & shale with highly variable bedding thicknesses

**Support :** 1.5 m x 15 mm spiral bars with full column resin in 22 mm diameter holes. Four bolts per row with rows 1.5 m apart.

**Layout : Depth** 55 to 60 m **Roadway** 6.0 m **Pillar** 6.0 m

**Mining :** Conventional drill and blast **Mining height** 2.1 m

***Figure 3-20 Colliery 'D' area 1 site 2 (roadway)***




**Notes**

**Coalfield** :Witbank **Seam** 2 seam **Position** Intersection

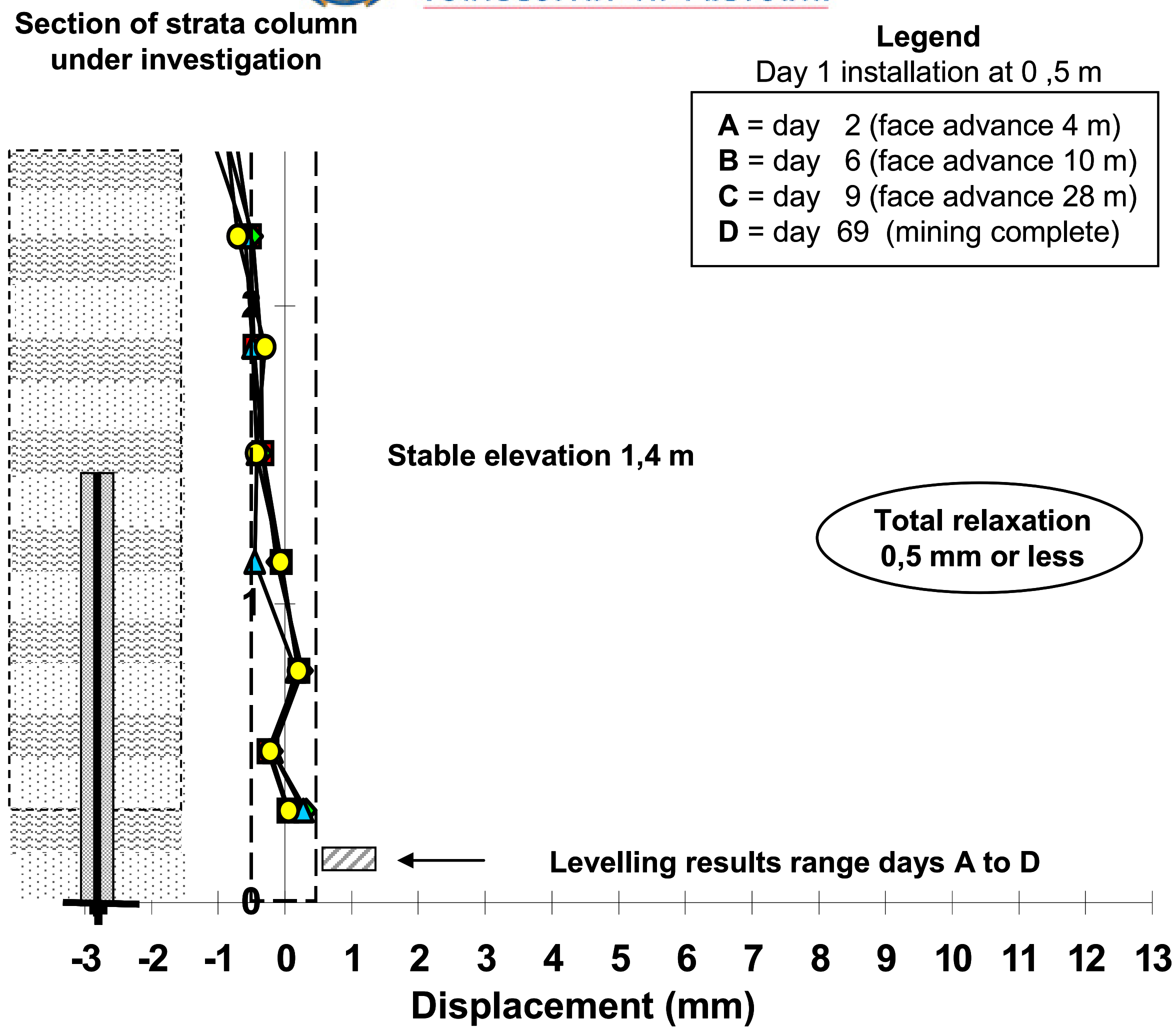
**Roof :** Laminated sandstone & shale with highly variable bedding thicknesses

**Support :** 1.5 m x 15 mm spiral bars with full column resin in 22 mm diameter holes. Four bolts per row with rows 1.5 m apart.

**Layout : Depth** 55 to 60 m **Roadway** 6.0 m **Pillar** 6.0 m

**Mining :** Conventional drill and blast **Mining height** 2.1 m

***Figure 3-21 Colliery 'D' area 1 site 3 (intersection)***




Notes

**Coalfield :**Witbank **Seam** 2 seam **Position** Roadway

**Roof :** Laminated sandstone & shale with highly variable bedding thicknesses

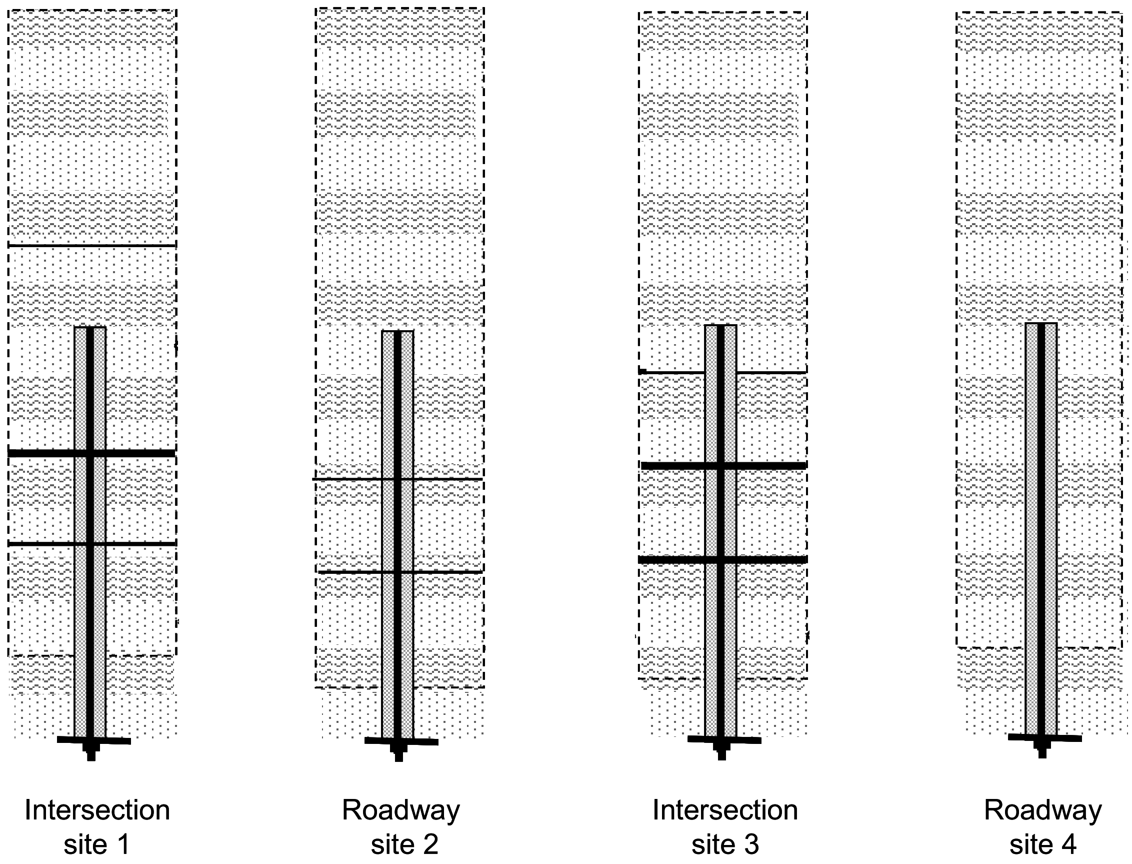
**Support :** 1,5 m x 15 mm spiral bars with full column resin in 22 mm diameter holes. Four bolts per row with rows 1,5 m apart.

**Layout : Depth** 55 to 60 m **Roadway** 6,0 m **Pillar** 6,0m

**Mining :** Conventional drill and blast **Mining height** 2,1m

*Figure 3-22 Colliery 'D' area 1 site 4 (roadway)*

**Witbank 2 Seam** **Colliery 'D'** **Drill and blast section**




**Roof :** Laminated sandstone and shale with highly variable bedding thicknesses

**Support :** 1.5 m x 15 mm diameter spiral bars in 22 mm diameter holes with full column resin. Four bolts per hole with rows 1.5 m apart.

***Figure 3-23 Colliery 'D' area 1 comparative roof behaviour***

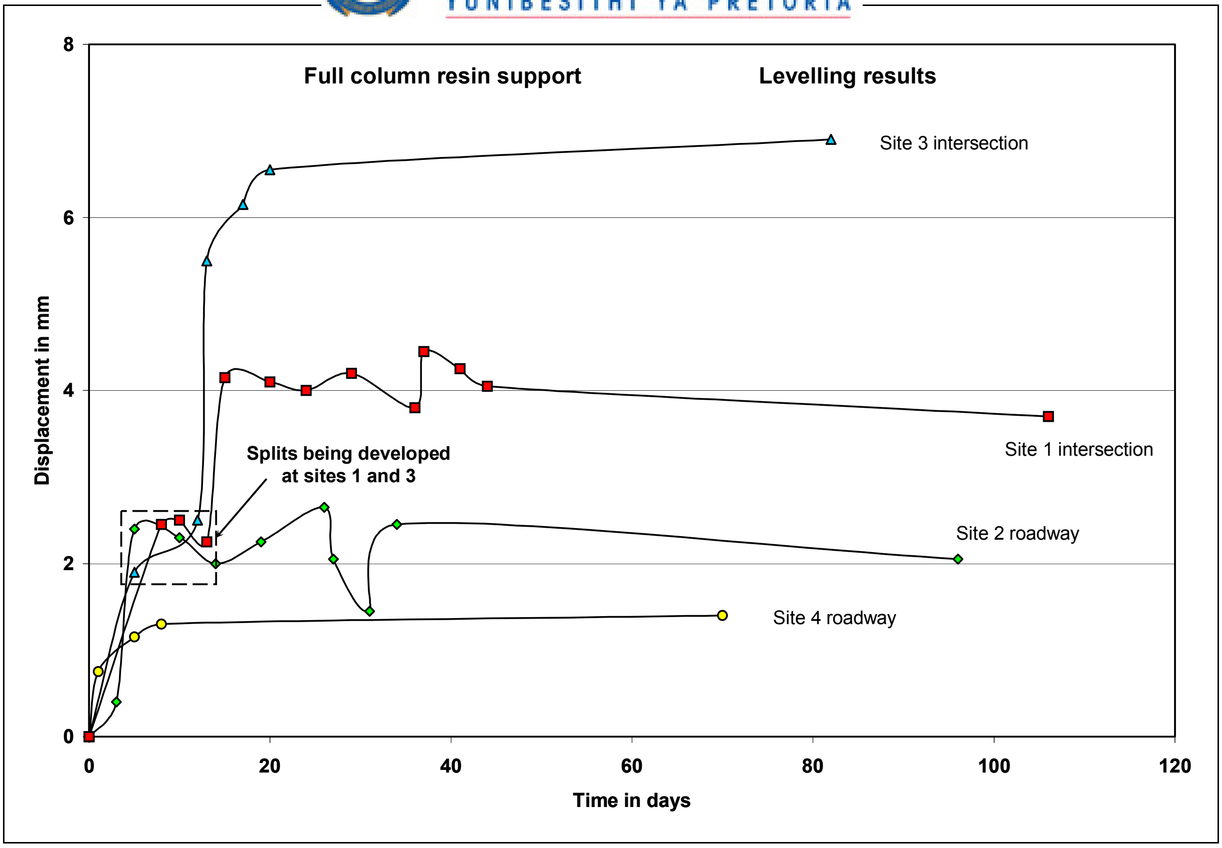



***Figure 3-24 Colliery 'D' area 1 comparison of roof skin displacement***

In the continuous miner (CM) cutting cycle approximately half the road width was mined for 7.0 m before the support was installed. The support consisted of two bolts per row with the rows 1.5 m apart. The adjacent side of the roadway was then mined up to the face after which the additional two bolts were added to each row.

At three of the four sites in area 2, the levelling results were close to the values indicated by the lowest anchor in the sonic probe string. The exception was site 3 (Figure 3-27) where the levelling results were nearly 4.0 mm larger, indicating the presence of additional displacements below the sonic probe bottom anchor.

In the intersection with the headboards, site 1 (Figure 3-25), there are indications of some small displacements in excess of 1.0 m above the bolt horizon, up to the 2.5 m elevation. There were large displacements totalling 9.0 mm in the 0.3 m immediate skin of the roof below the resin column. These displacements occurred relatively quickly after the splits were mined. Contributing factors could have included a lack of stiffness of the headboard and irregular contact with the roof. However, since 80 per cent of the total displacement took place within the first four days, a more likely cause could have been insufficient tension applied to the bolts during installation. Timber shrinkage with time is unlikely to have had any real effect over such a short time period.

In the intersection at site 2 (Figure 3-26), the total relaxation of 12 mm, at the lowest anchor was identical to site 1, as was the stable elevation at 2.5 m into the roof. The behaviour of the roof strata was however completely different. The displacements within the 2.5 m zone tended to be more linear. The resin column within the bolt horizon appeared to be ineffective as the two largest displacements occurred within this region. A large portion, approximately 75 per cent, of the final displacement occurred within 24 hours when the face had advanced 7.0 m and the first split had holed through.

When compared to the other two intersections at sites 3 and 4 (Figure 3-27 and Figure 3-28) with the same support systems, the site 2 (Figure 3-26) intersection roof strata was by far the most active. The apparent ineffectiveness of the resin bond column suggests that the support may not have been correctly installed. The levelling results agree fairly closely with the sonic probe bottom anchor, with the exception of day 2, which is so far out that it is in all probability an erroneous reading.

The overall roof strata behaviour at site 3 (Figure 3-27) was similar to that at site 1. The largest displacements were below the resin column. The levelling results indicate the presence of additional displacements of approximately 4.0 mm situated within 0.1 to 0.2 m of the immediate roof skin. This is not included on the strata column diagram and would increase the total relaxation to at least 10 mm. Unlike sites 1 and 2, there is no evidence of displacements above the 1.9 m elevation.

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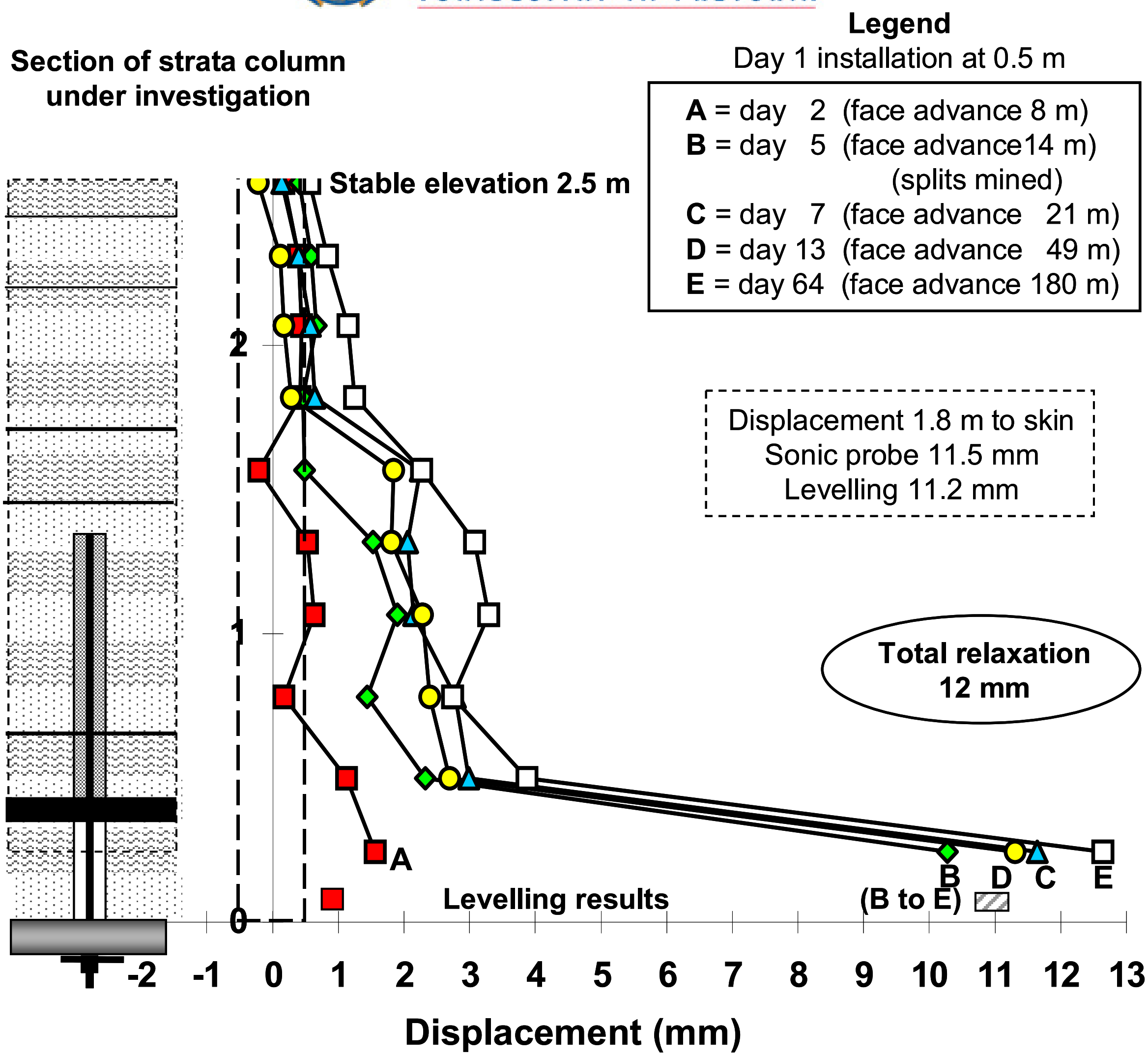



**Notes**

**Coalfield :** Witbank **Seam** 2 seam **Position** Intersection

**Roof :** Laminated sandstone & shale with highly variable bedding thicknesses.

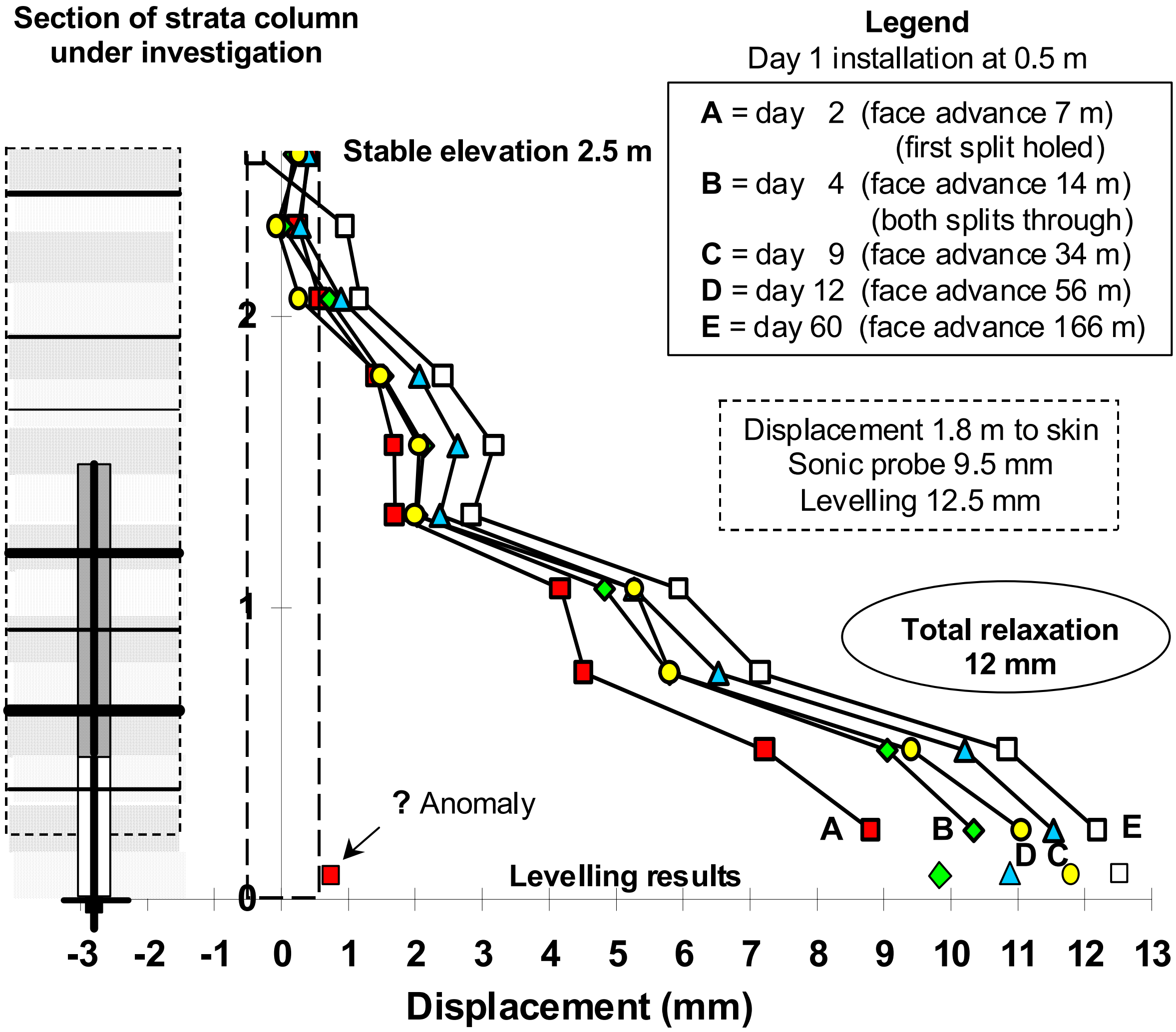
**Support :** 1.5 m x 15 mm spiral bars with partial column resin with head boards in 22 mm diameter holes. Four bolts per row with rows 1.5 m apart.

**Layout : Depth** 55 to 60 m **Roadway** 6.0 m **Pillar** 6.0 m

**Mining :** Continuous miner **Mining height** 2.1 m

Although there appear to be some small displacements above the bolt horizon, there are large displacements below the resin column, most likely as a result of the lack of stiffness of the headboard or insufficient tension applied to the tendon.

***Figure 3-25 Colliery 'D' area 2 site 1 (intersection)***




## Notes

**Coalfield :** Witbank **Seam** 2 seam **Position** Intersection

**Roof :** Laminated sandstone & shale with highly variable bedding thicknesses.

**Support :** 1.5 m x 15 mm spiral bars with partial column resin in 22 mm diameter holes. Four bolts per row with rows 1.5 m apart.

**Layout : Depth** 55 to 60 m **Roadway** 6.0 m **Pillar** 6.0 m

**Mining :** Continuous miner **Mining height** 2.1 m

The displacements extend to above the bolt horizon. The relatively large displacements recorded within the resin column horizon indicate that the support system was ineffective and suggest that the support may not have been correctly installed.

***Figure 3-26 Colliery 'D' area 2 site 2 (intersection)***

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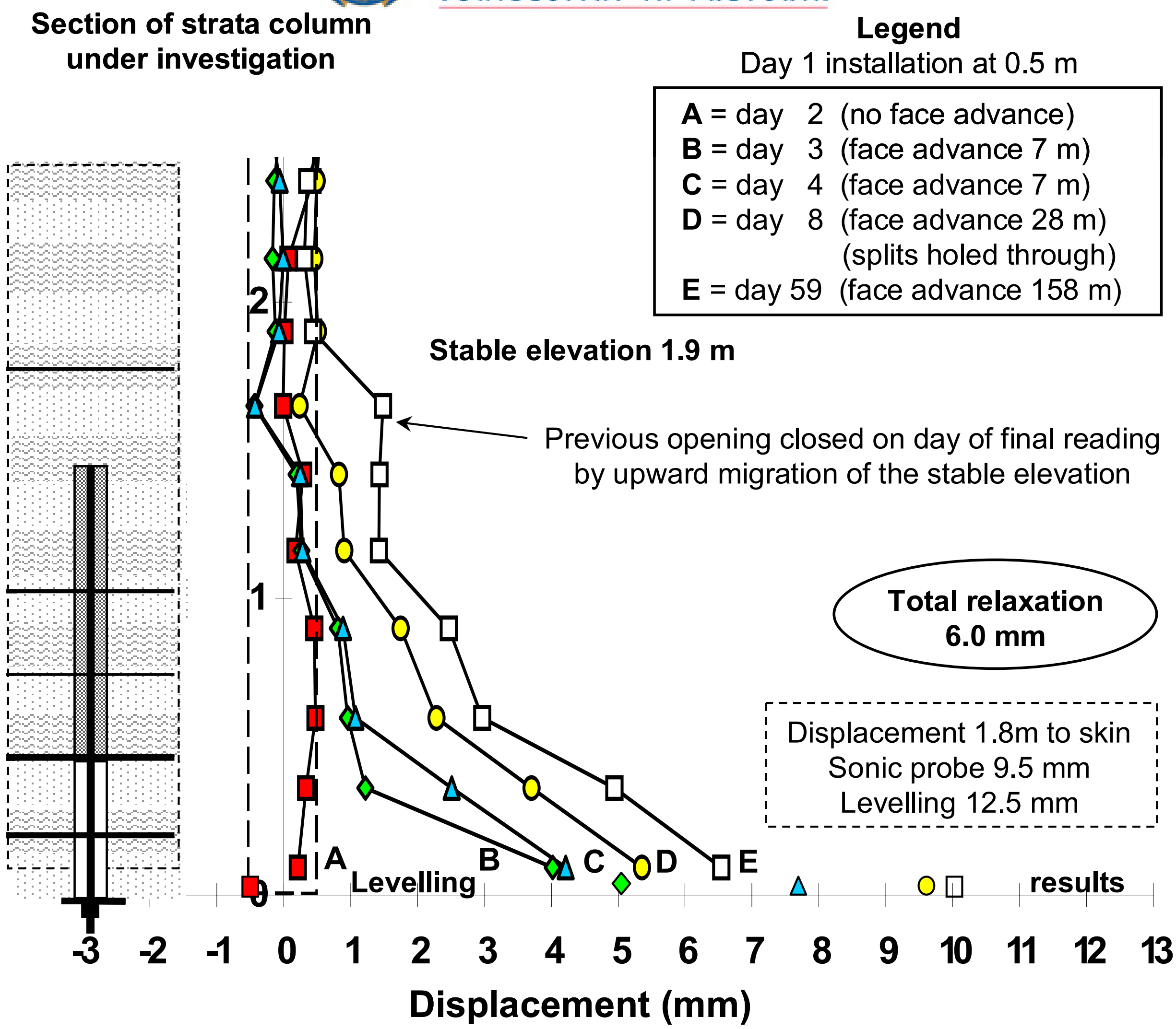



**Notes**

**Coalfield :** Witbank **Seam** 2 seam **Position** Intersection

**Roof :** Laminated sandstone & shale with highly variable bedding thicknesses.

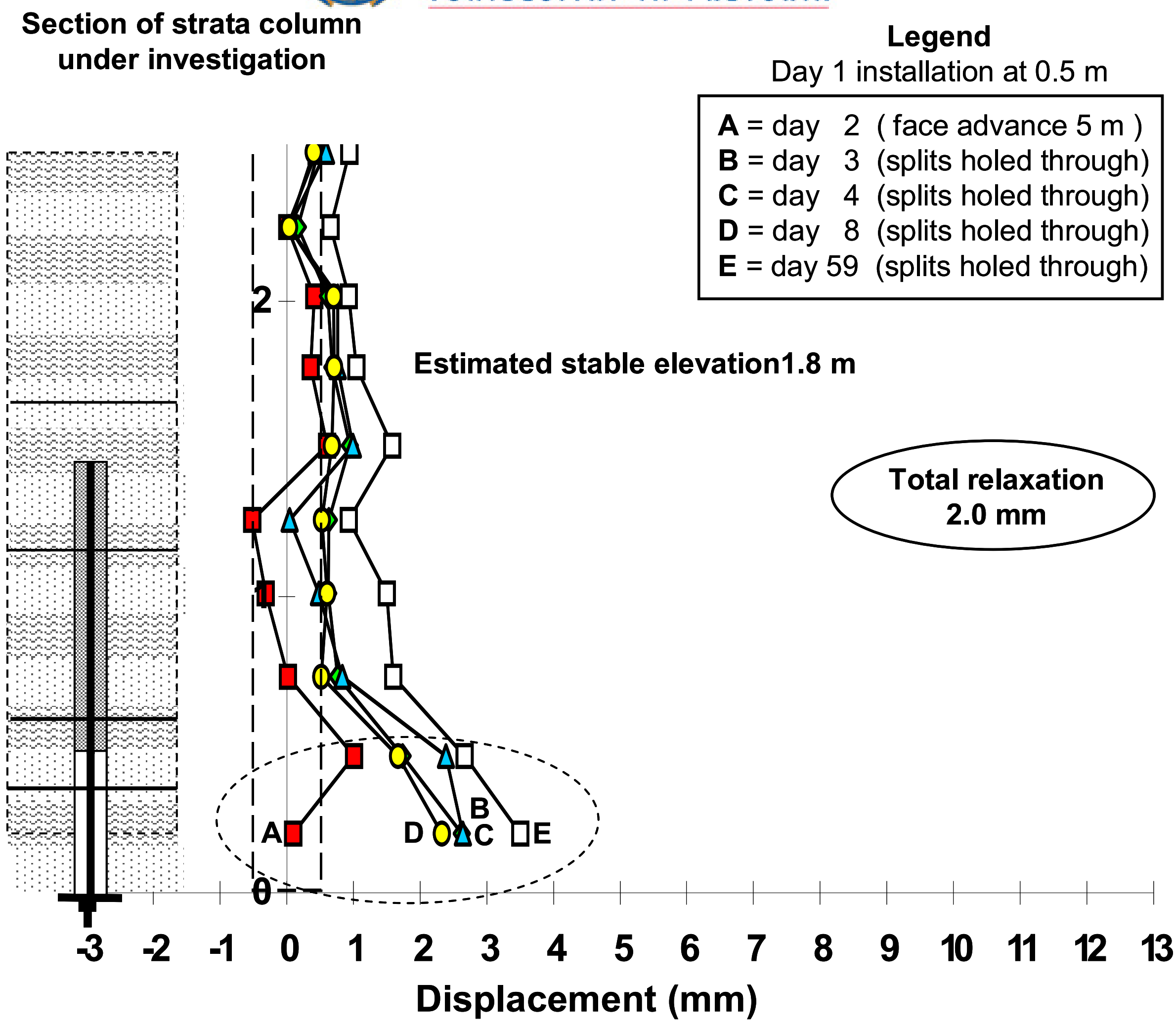
**Support :** 1.5 m x 15 mm spiral bars with partial column resin in 22 mm diameter holes. Four bolts per row with rows 1.5 m apart.

**Layout : Depth** 55 to 60 m **Roadway** 6.0 m **Pillar** 6.0 m

**Mining :** Continuous miner **Mining height** 2.1 m

Although the displacements extend to about 0.3 m above the bolt horizon. Most of the displacement occurred close to the roof skin below the resin column.

*Figure 3-27 Colliery ‘D’ area 2 site 3 (intersection)*




**Notes**

**Coalfield :**Witbank **Seam** 2 seam **Position** Intersection

**Roof :** Laminated sandstone & shale with highly variable bedding thicknesses.

**Support :** 1.5 m x 15 mm spiral bars with partial column resin in 22 mm diameter holes. Four bolts per row with rows 1.5 m apart.

**Layout : Depth** 55 to 60 m **Roadway** 6.0 m **Pillar** 6.0 m

**Mining :** Continuous miner **Mining height** 2.1 m

The displacements extended to about 0.3 m above the bolt horizon.

***Figure 3-28 Colliery 'D' area 2 site 4 (intersection holed into)***

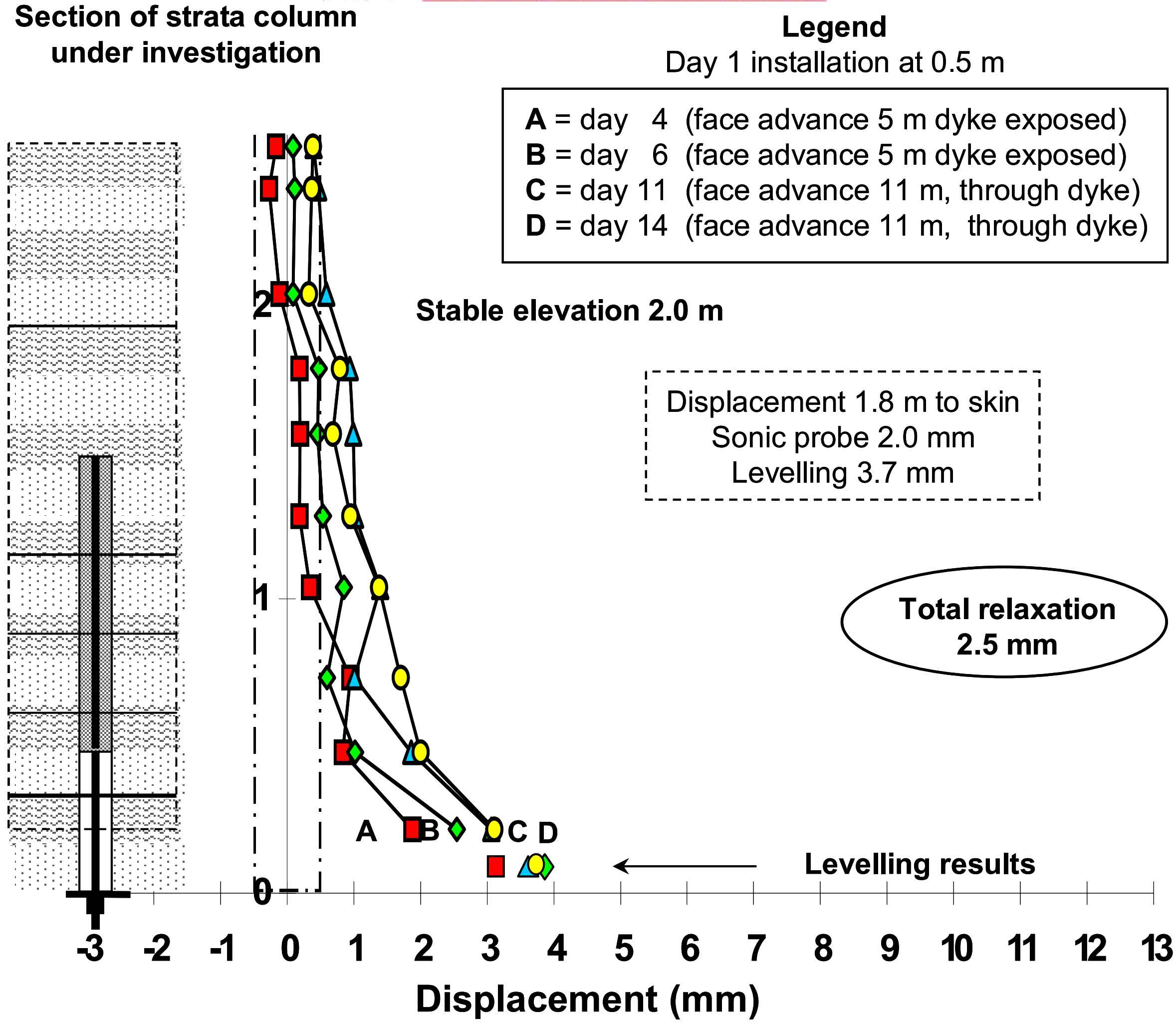
The displacements were limited to the 1.8 m elevation and the total relaxation, recorded at 2.0 mm, was the lowest of all four intersections. However, since there was no levelling backup at this site, it is not known if there were additional displacements in the immediate roof below the bottom sonic probe anchor.

Site 5 of area 2 (Figure 3-29) was in a roadway approaching a dyke. The road width was reduced to approximately 5.0 m, which resulted in the roof bolts being closer together in the rows. Although there appears to be small displacements up to the 2.0 m elevation, the majority of the displacements were within the initial 1.2 m of roof strata and were 2.5 mm in total. Although slightly higher in value, the levelling results agreed fairly closely with the bottom sonic probe anchor.

For comparison purposes the results of all five sites are grouped together in Figure 3-30. The strata performance at sites 3, 4 and 5 were similar showing the roof to have been active below the 2.0 m elevation, approximately 0.5 m above the bolted zone. As previously mentioned, the roof behaviour at the intersections at sites 1 and 2 produced larger displacements than at the intersection at site 3. Headboards were used in site 1 and the quality of support installation at site 2 is suspect.

The total relaxation within the initial 1.8 m of roof strata, as recorded by the levelling results at four of the five sites, have been plotted together and are presented in Figure 3-31. The overall relaxation at sites 1 and 2 were approximately 10 and 20 per cent higher than the site 3 values.

The last site, site 5, investigated in area 2 was in a continuous miner section. The support pattern remained the same. The support system was changed to 1.5 m x 18 mm rebar installed in the smallest hole diameter of 22 mm, using two 19 mm x 380 mm resin cartridges. This resulted in full column resin support. The difference between this support and the support installed in area 1, apart from the increase in the cross sectional area of the steel tendon by approximately 26 per cent, was the use of 200 x 200 mm dome washers in place of the usual 150 x 150 mm washers.




## Notes

**Coalfield :** Witbank **Seam** 2 seam **Position** Roadway

**Roof :** Laminated sandstone & shale with highly variable bedding thicknesses.

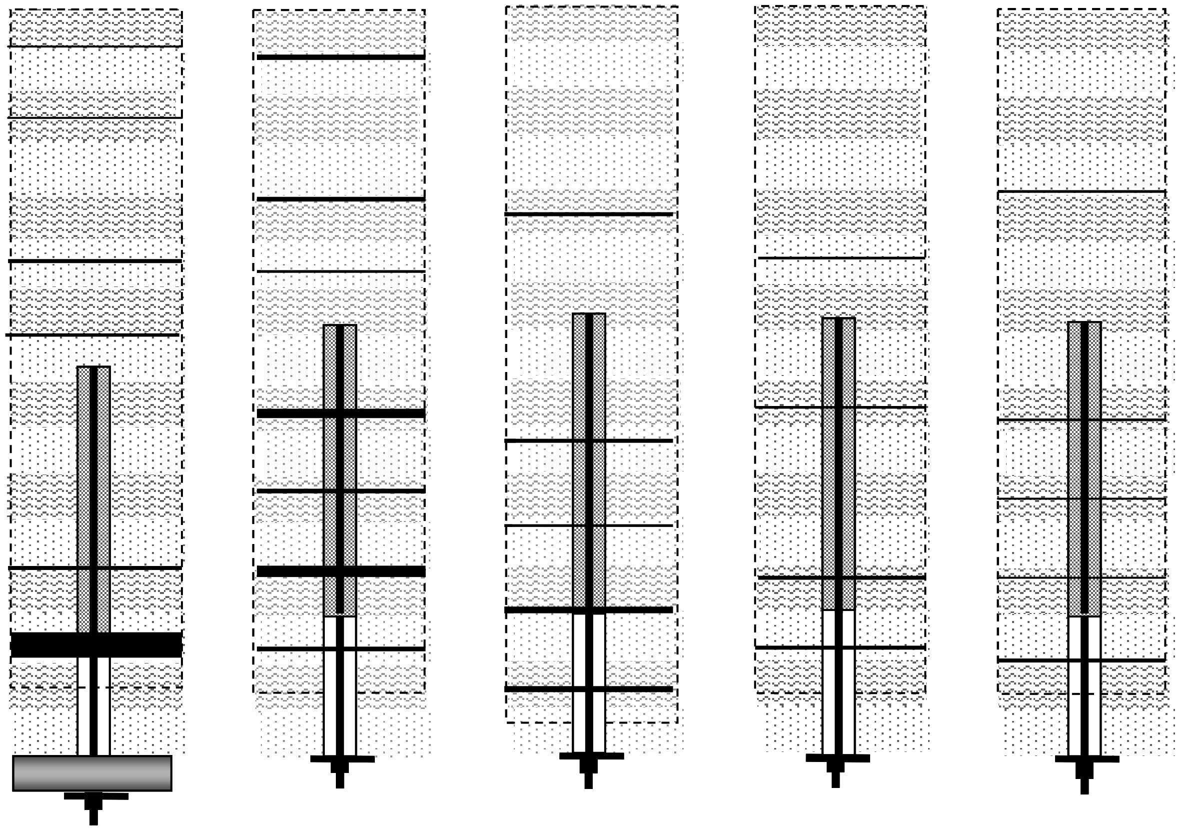
**Support :** 1.5 m x 15 mm spiral bars with partial column resin in 22 mm diameter holes. Four bolts per row with rows 1.5 m apart.

**Layout : Depth** 55 to 60 m **Roadway** 5.0 m **Pillar** 6.0 m

**Mining :** Continuous miner **Mining height** 2.1 m

The displacements extended to about 0.4 m above the bolt horizon.

*Figure 3-29 Colliery 'D' area 2 site 5 (roadway approaching dyke)*




**Roof :** Laminated sandstone and shale with highly variable bedding thicknesses

**Support :** 1.5 m x 15 mm diameter spiral bars in 22 mm diameter holes with partial column resin. Four bolts per row with rows 1.5 m apart.

***Figure 3-30 Colliery 'D' area 2 comparative roof behaviour***

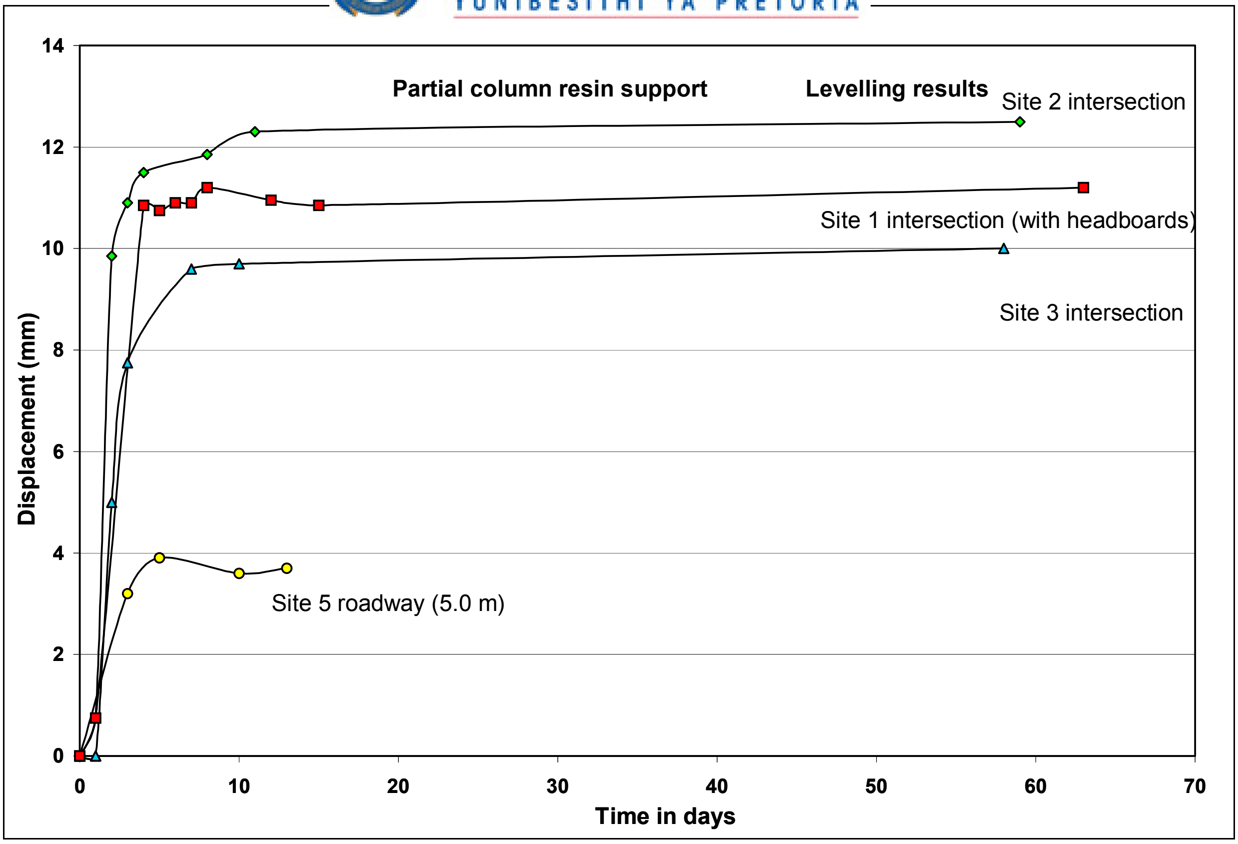



***Figure 3-31 Colliery 'D' area 2 comparison between roadway and intersection roof skin displacement***

The typical set of three sites was monitored, one intersection and two adjacent roadways. At site 3 after the installation of the instrumentation, the blind end of the roadway was not advanced. It was holed into from the other side. There was no backup levelling at any of the three sites.

There was very little displacement recorded at any of the sites as indicated in Figure 3-32, Figure 3-33 and Figure 3-34. The difference between the roof behaviour in the roadways and the intersection was hardly discernible. The stable elevation of the intersection increased slightly from on average less than 1.0 m in the roadways to about 1.2 m with a total relaxation of 1.0 mm. All the displacements recorded were well within the bolted zone. The comparative roof behaviour of the three sites in area 3 is presented in Figure 3-35.

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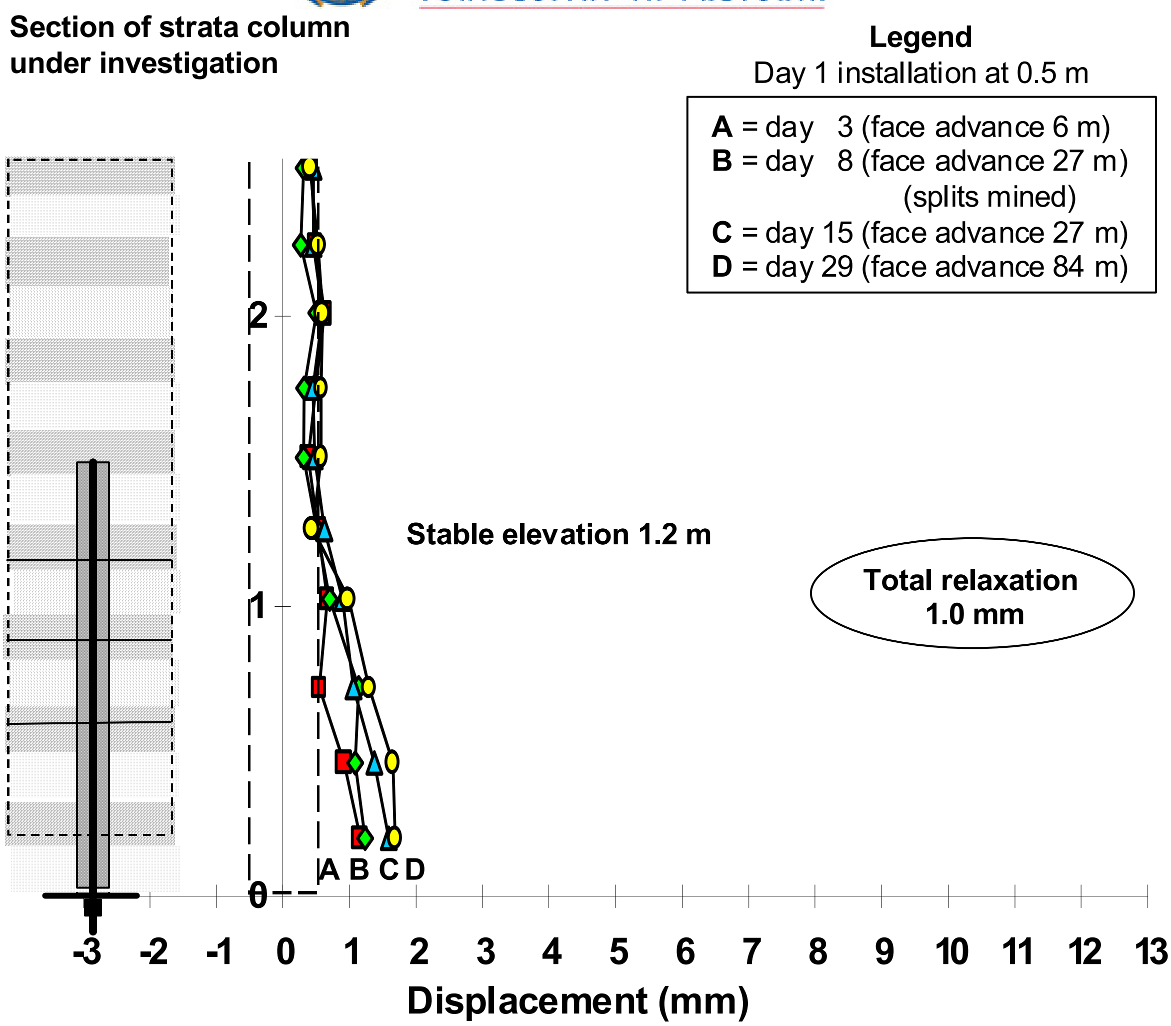



**Notes**

**Coalfield :** Witbank **Seam** 2 seam **Position** Intersection

**Roof :** Laminated sandstone & shale with highly variable bedding thicknesses.

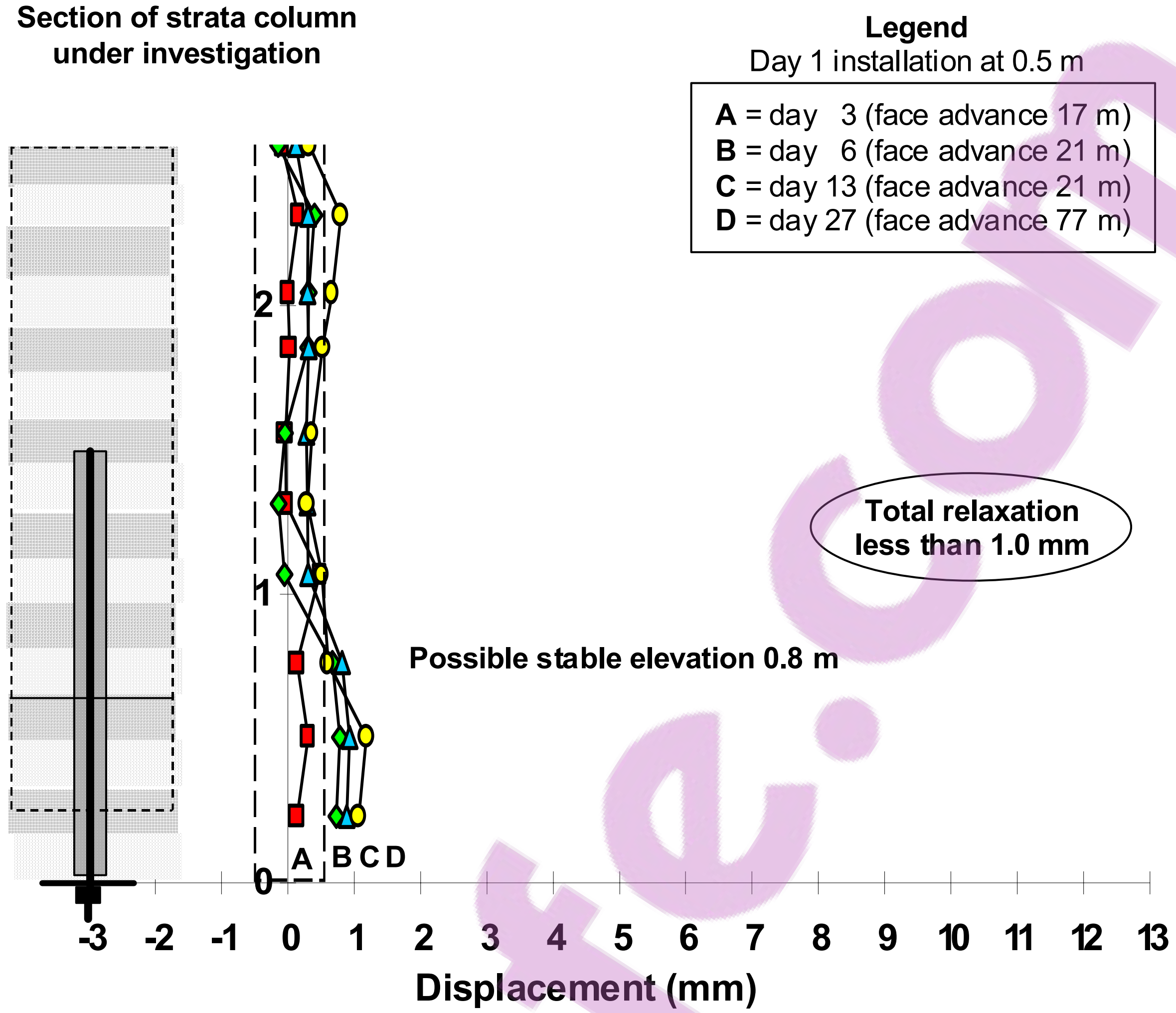
**Support :** 1.5 m x 18 mm rebars with 200 mm x 200 mm dome washers
full column resin in 22 mm diameter holes.
Four bolts per row with rows 1.5 m apart.

**Layout : Depth** 55 to 60 m **Roadway** 6.0 m **Pillar** 6.0 m

**Mining :** Continuous miner **Mining height** 2.1 m

Very small displacements all within the bolt horizon.

*Figure 3-32 Colliery 'D' area 3 site 1 (intersection)*




**Notes**

**Coalfield :** Witbank **Seam** 2 seam **Position** Roadway

**Roof :** Laminated sandstone & shale with highly variable bedding thicknesses.

**Support :** 1.5 m x 18 mm rebars with 200 mm x 200 mm dome washers
full column resin in 22 mm diameter holes.
Four bolts per row with rows 1.5 m apart.

**Layout : Depth** 55 to 60 m **Roadway** 6.0 m **Pillar** 6.0 m

**Mining :** Continuous miner **Mining height** 2.1 m

Hardly any displacements recorded.

*Figure 3-33 Colliery 'D' area 3 site 2 (roadway)*

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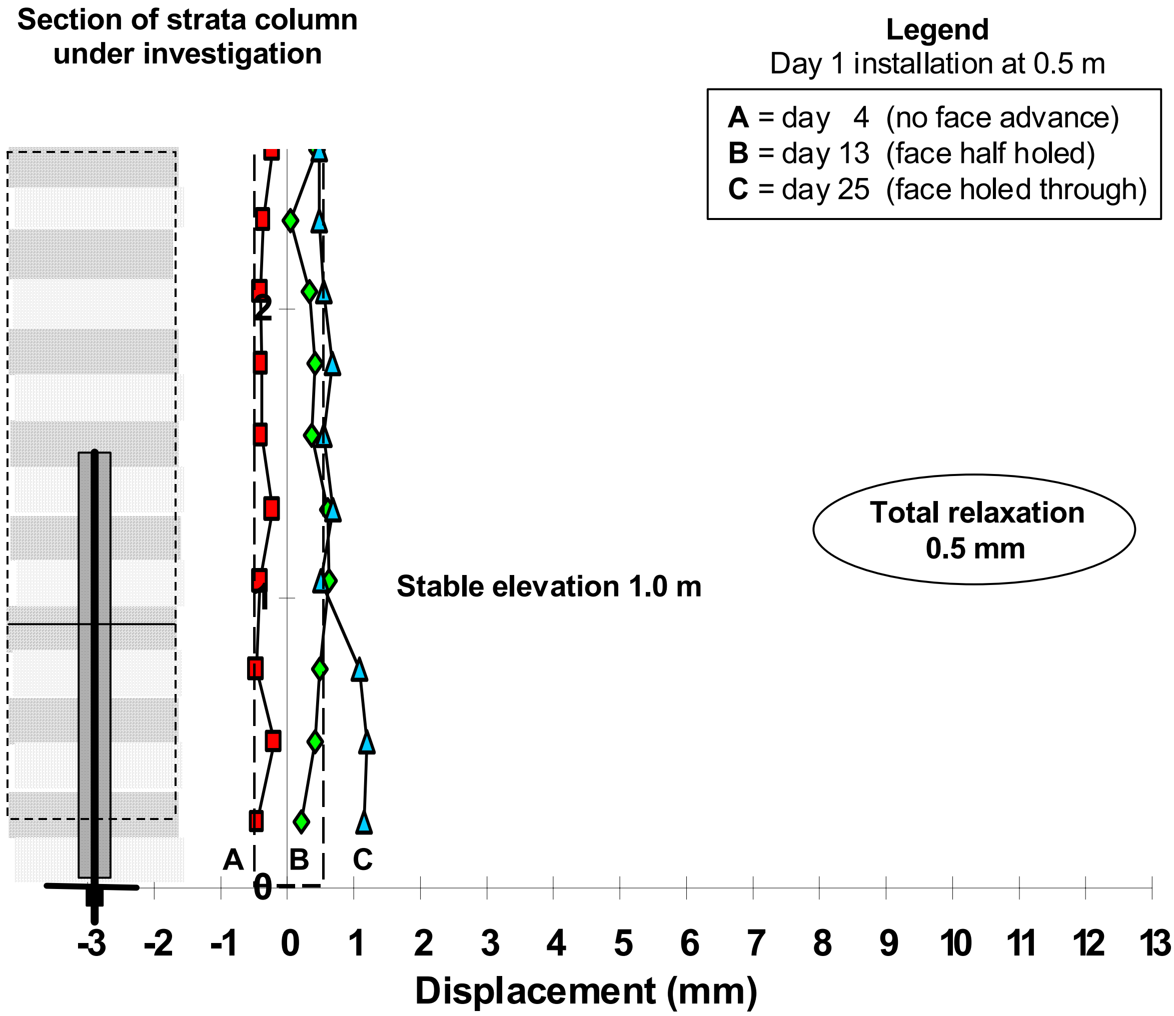



**Notes**

**Coalfield :** Witbank **Seam** 2 seam **Position** Roadway

**Roof :** Laminated sandstone & shale with highly variable bedding thicknesses.

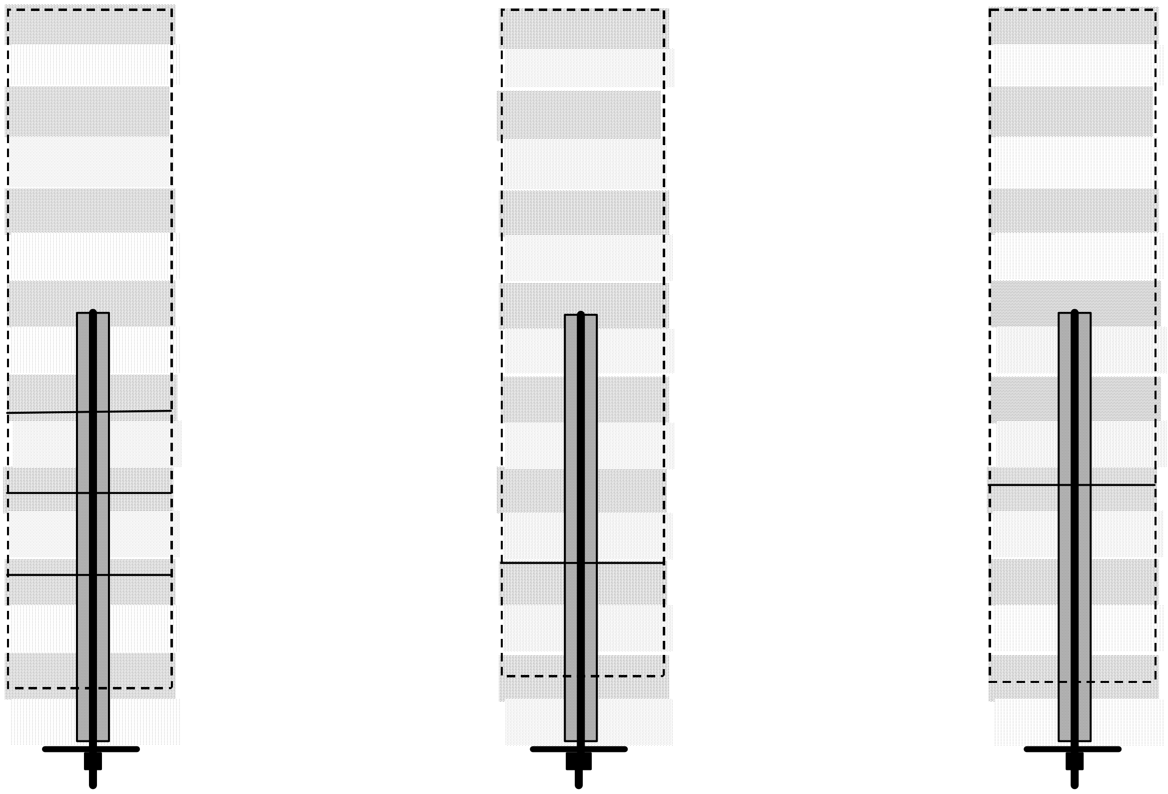
**Support :** 1.5 m x 18 mm re-bars with 200 mm x 200 mm dome washers
full column resin in 22 mm diameter holes.
Four bolts per row with rows 1.5 m apart.

**Layout : Depth** 55 to 60 m **Roadway** 6.0 m **Pillar** 6.0 m

**Mining :** Continuous miner **Mining height** 2.1 m

Hardly any displacements recorded.

***Figure 3-34 Colliery 'D' area 3 site 3 (roadway blind end holed into)***




**Roof :** Laminated sandstone and shale with highly variable bedding thicknesses

**Support :** 1.5 m x 18 mm diameter re- bars in 22 mm diameter holes with full column resin and 200 mm x 200 mm dome washers. Four bolts per row with rows 1.5 m apart.

***Figure 3-35 Colliery 'D' area 3 comparative roof behaviour***

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### 3.7.1 Site performance summary Colliery 'D' area 1

| | | | |
|---|---|---|---|
| Coalfield: | Witbank | Seam: | 2 Seam |
| Sites: | Four | Positions: | Two intersections two roadways |
| Road widths: | 6.0 m | Pillar widths: | 6.0 m |
| Mining height: | 2.1 m | Depth: | 50 to 60 m |

Mining method: Drill and blast

Roof strata: Laminated sandstone and shale with highly variable bedding thicknesses.

Support: 1.5 m x 15 mm diameter spiral bars with full column resin in 22 mm diameter holes. Four bolts per row with rows 1.5 m apart.

Performance: Displacement occurred in the roof strata when the face was advanced with the blast, and the unsupported roof span was increased up to a maximum of 3.5 m. This resulted in open parting planes or fractures being present as close as 0.5 m from the face prior to the installation of the support and instrumentation. These openings appeared to be mainly within the immediate 0.4 m of the roof. By and large, the roof displacements were contained within the bolt horizon.

### 3.7.2 Site performance summary Colliery 'D' area 2

| | | | |
|---|---|---|---|
| Coalfield: | Witbank | Seam: | 2 Seam |
| Sites: | Five | Positions: | Four intersections one roadway |
| Road widths: | 6.0 m<br>(5.0 m site 5) | Pillar widths: | 6.0 m |
| Mining height: | 2.1 m | Depth: | 50 to 60 m |

Mining method: Continuous miner

Roof strata: Laminated sandstone and shale with highly variable bedding thicknesses.

Support: 1.5 m x 15 mm diameter spiral bars with partial column resin in 22 mm diameter holes. Four bolts per row with rows 1.5 m apart.

Performance: In general, there were small displacements above the bolt horizon up to the 2.0 m to 2.5 m elevations. There were also small displacements within the resin bond horizon with somewhat larger displacements in the unbounded

0.3 m to 0.4 m of immediate roof. The exception was site 2 where the major displacements were within the resin bond horizon. The overall performance of the only intersection with headboards indicated that 75 per cent of the displacements were within the 0.3 m of unbounded immediate roof

### 3.7.3 Site performance summary Colliery 'D' area 3

| | | | |
|---|---|---|---|
| Coalfield: | Witbank | Seam: | 2 Seam |
| Sites: | Three | Positions: | One intersection two roadways |
| Road widths: | 6.0 m | Pillar widths: | 6.0 m |
| Mining height: | 2.1 m | Depth: | 50 to 60 m |

Mining method: Continuous miner

Roof strata: Laminated sandstone and shale with highly variable bedding thicknesses.

Support: 1.5 m x 18 mm diameter rebars with full column resin in 22 mm diameter holes. Four bolts per row with rows 1.5 m apart.

Performance: There were only a few small displacements all contained within the bolt horizon.

## 3.8 Colliery 'E '

Colliery 'E' was the last colliery investigated. Five sites were monitored in the gateroads and associated splits at the edge of a shortwall panel. The mining was carried out by continuous miner. The laminated sandstone roof was supported by 1.8 m long 16 mm diameter full column resin rebar. The support pattern was four bolts per row with the bolts 1.0 m apart and 1.5 m between rows. Although backup levelling was attempted, it proved impractical to monitor due to the installation of the belt and the dumping of rubble. In addition to monitoring the effects of the development of the roadways, attempts were also made to monitor the dynamic effects of the approaching shortwall face.

Overall, very little if any displacement was recorded as the roof remained stable throughout the mining process. The only site to record what appeared to be a real displacement was site 1, Figure 3-36, where a total relaxation of approximately 1.0 mm occurred within 0.3 m of the roof skin. The results from the other four sites are presented in Figure 3-37, Figure 3-38, Figure 3-39 and Figure 3-40.

#### 3.8.1.1 Site performance summary Colliery ‘E’

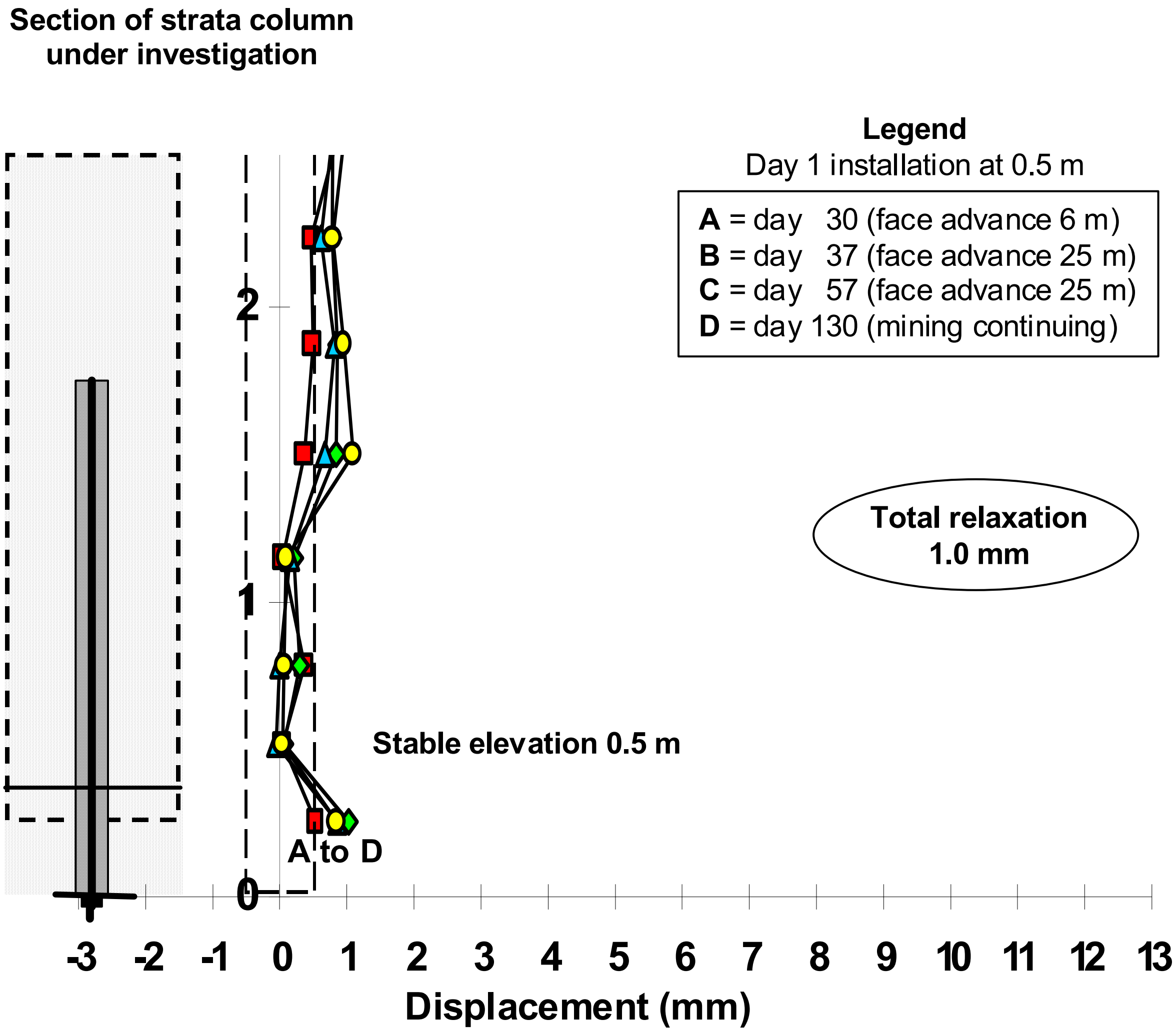
| | | | |
|---|---|---|---|
| Coalfield: | Highveld | Seam: | 2 Seam |
| Sites: | Five | Positions: | Four roadway one split |
| Road widths: | 6.5 m | Pillar widths: | Chain pillar 50 m centres |
| Mining height: | 3.0 m | Depth | 50 m |

Mining method: Continuous miner

Roof strata: Laminated sandstone.

Support: 1.8 m x 16 mm diameter rebar with full column resin. Four bolts per row with bolts 1.0 m apart and 1.5 m between rows.

Performance: Only one site recorded a very slight relaxation of 1.0 mm.




**Notes**

**Coalfield :Highveld** **Seam** 2 seam **Position** Shortwall gate road

**Roof :** Laminated sandstone

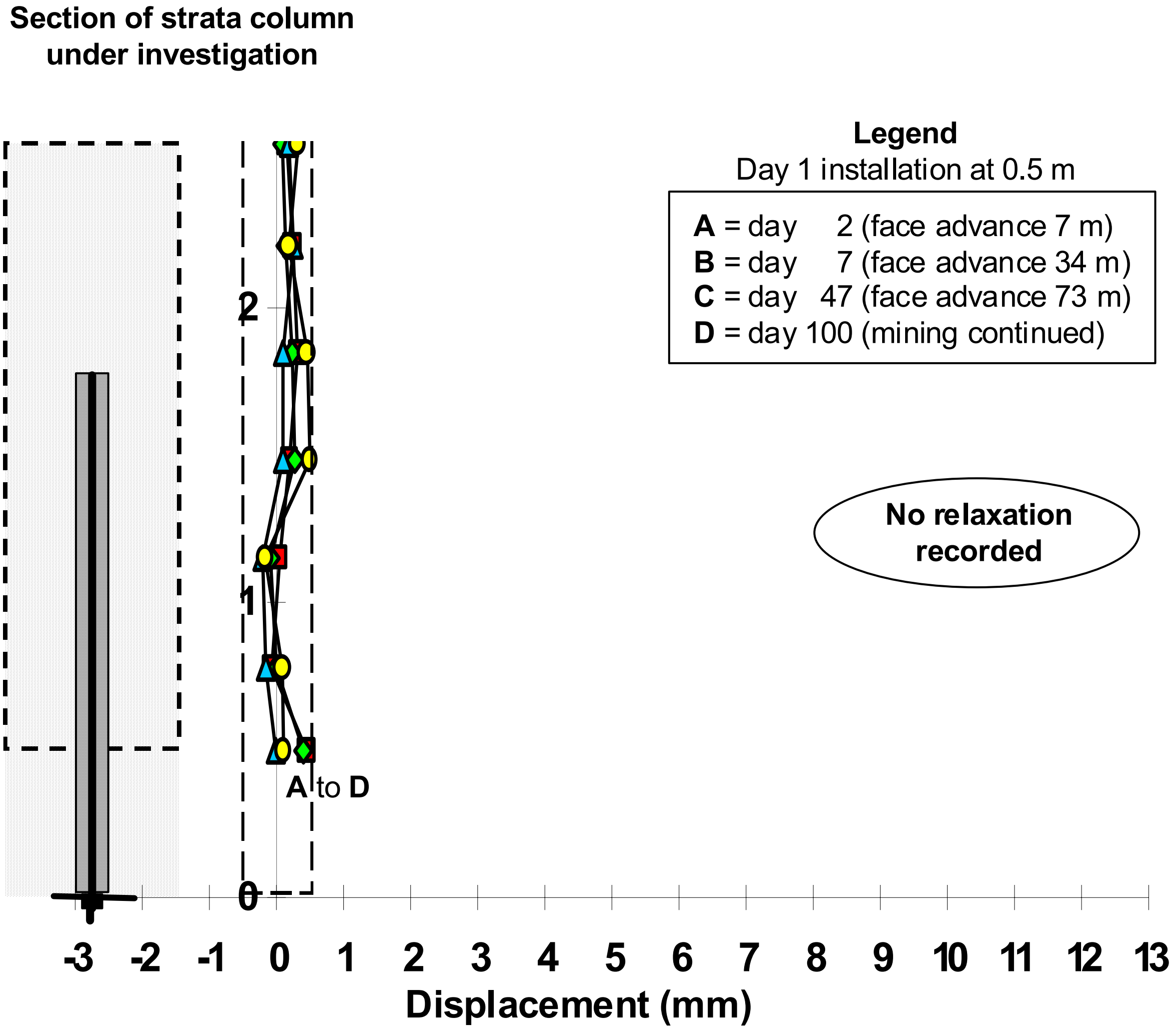
**Support :**1.8 m x 16 mm full column resin bolts four in a row (1.0 m apart) across the roadway, 1.5 m between rows.

**Layout : Depth** 50 m **Roadway** 6.5 m **Pillars** chain pillars 50 m centres

**Mining :** Continuous miner **Mining height** 3.0 m

Very small displacements were recorded

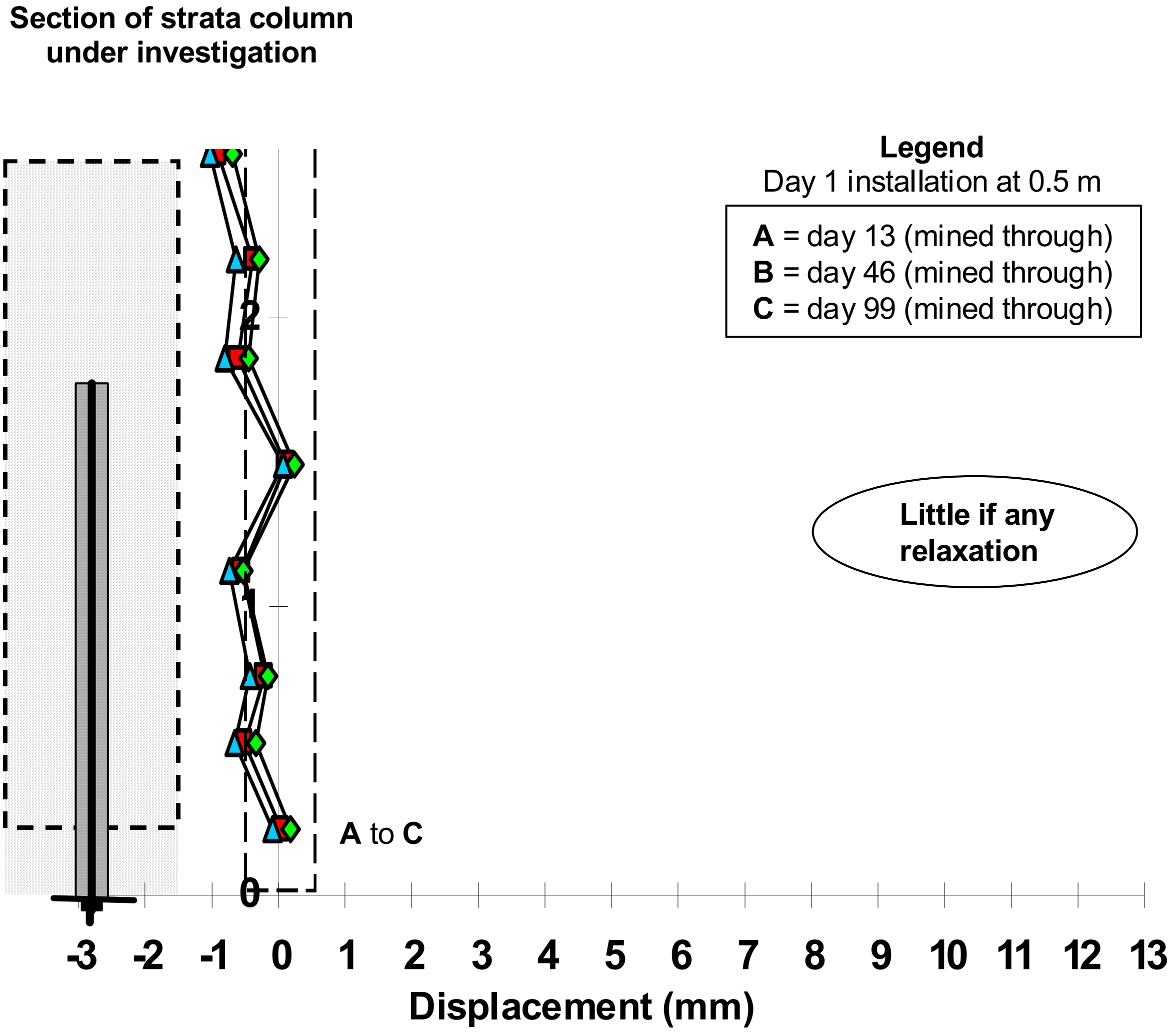
***Figure 3-36 Colliery 'E' site 1 (gate road)***




**Notes**

| |
|---|
| **Coalfield :Highveld** **Seam** 2 seam **Position** Shortwall gate road |
| **Roof :** Laminated sandstone |
| **Support :**1.8 m x 16 mm full column resin bolts four in a row (1.0 m apart) across the roadway, 1.5 m between rows. |
| **Layout : Depth** 50 m **Roadway** 6.5 m **Pillars** chain pillars 50 m centres |
| **Mining :** Continuous miner **Mining height** 3.0 m |
| No displacements were recorded |

*Figure 3-37 Colliery 'E' site 2 (gate road)*




**Notes**

**Coalfield :Highveld** **Seam** 2 seam **Position** Shortwall gate road split

**Roof :** Laminated sandstone

**Support :**1.8 m x 16 mm full column resin bolts four in a row (1.0 m apart) across the roadway, 1.5 m between rows.

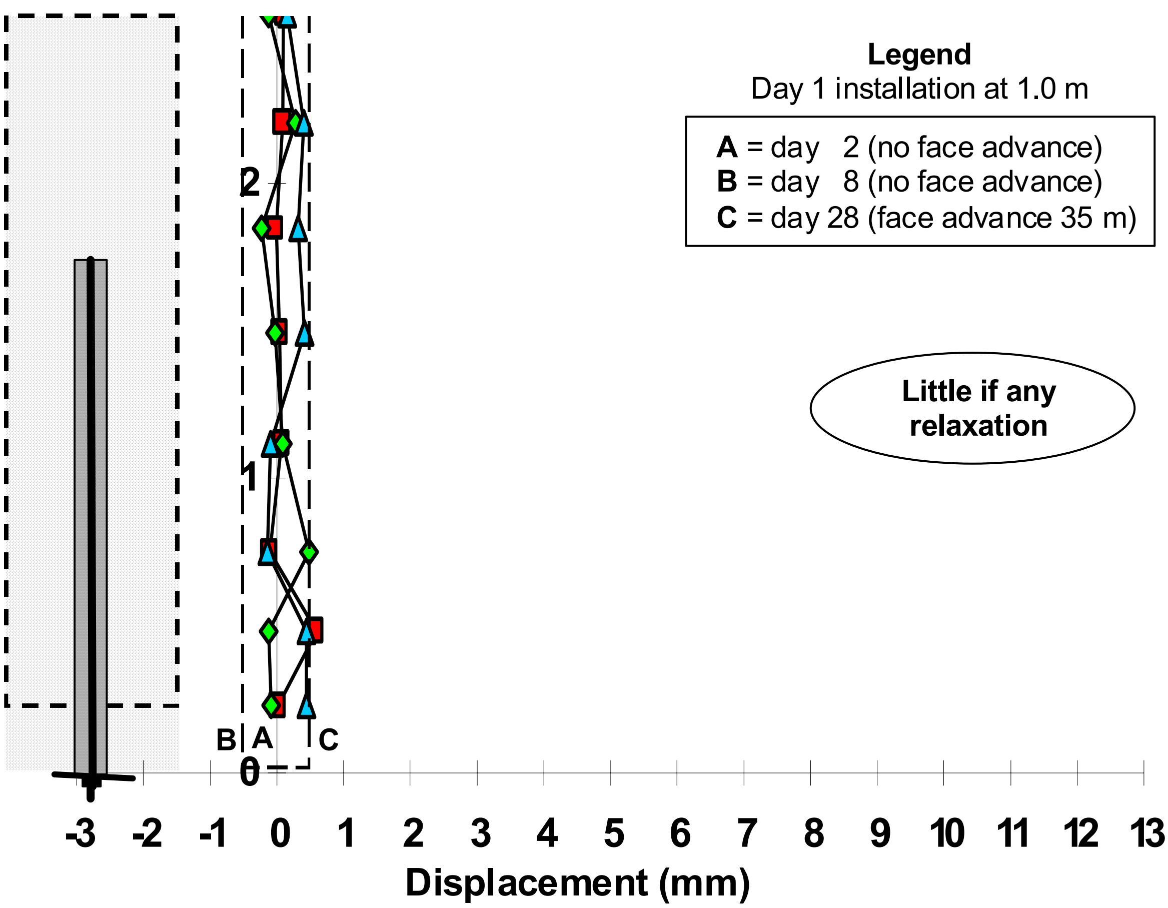
**Layout : Depth** 50 m **Roadway** 6.5 m **Pillars** chain pillars 50 m centres

**Mining :** Continuous miner **Mining height** 3.0 m

No displacements were recorded

*Figure 3-38 Colliery 'E' site 3 (split between gate roads)*

Section of strata column under investigation




**Notes**

**Coalfield :Highveld** **Seam** 2 seam **Position** Shortwall gate road

**Roof :** Laminated sandstone

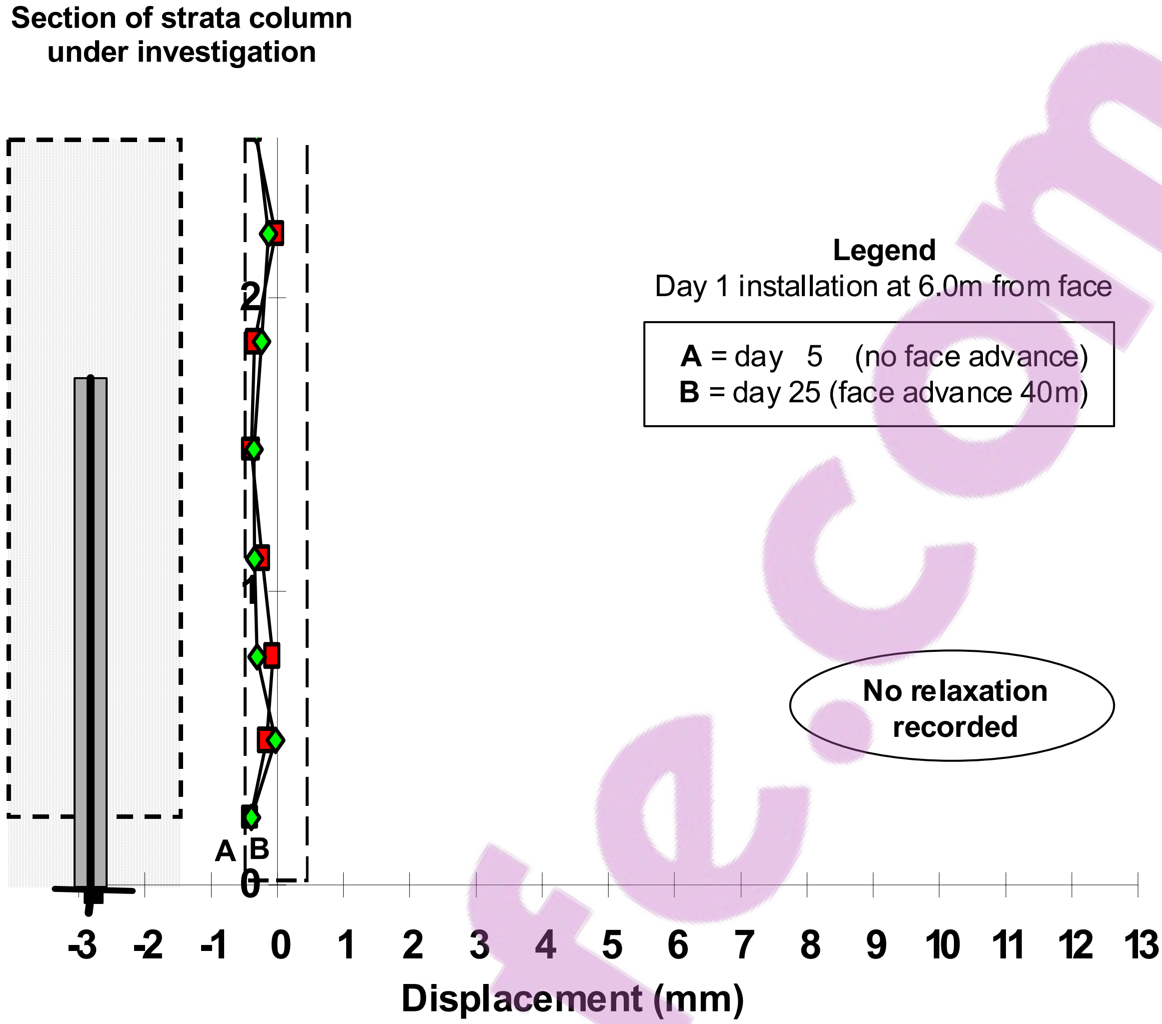
**Support :**1.8 m x 16 mm full column resin bolts four in a row (1.0 m apart) across the roadway, 1.5 m between rows.

**Layout : Depth** 50 m **Roadway** 6.5 m **Pillars** chain pillars 50 m centres

**Mining :** Continuous miner **Mining height** 3.0 m

No displacements were recorded

*Figure 3-39 Colliery 'E' site 4 (gate road)*




**Notes**

**Coalfield :Highveld** **Seam** 2 seam **Position** Shortwall gate road

**Roof :** Laminated sandstone

**Support :**1.8 m x 16 mm full column resin bolts four in a row (1.0 m apart) across the roadway, 1.5 m between rows.

**Layout : Depth** 50 m **Roadway** 6.5 m **Pillars** chain pillars 50 m centres

**Mining :** Continuous miner **Mining height** 3.0 m

No displacements were recorded

*Figure 3-40 Colliery 'E' site 5 (roadway)*



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## 3.9 Analysis of underground field measurements

In Table 3-1 the roof strata, mining method, support type and monitoring position of all the underground sites at the five collieries are listed. At Colliery 'E', where four of the sites were completely stable, they have been recorded in a single line as "Roadway x 4". The background information with regard to the sonic probe results, levelling measurements and the stable roof elevations are also listed.

For comparison purposes the roadway and intersection information at each colliery or area within a colliery has been averaged separately. The stable roof elevations have been averaged in a similar manner. A breakdown of the displacements recorded between the 1.8 m elevation and the lowest sonic probe anchor has been calculated for a more accurate comparison with the levelling results.

In Table 3-2 comparisons are made between the intersections and roadways with regard to the total relaxation and stable roof elevations. The percentage increase in the values recorded at the intersections, in relation to the roadways, were calculated and varied between 25 and 460 per cent. The overall average of the five different areas was 197 per cent. This indicates that, for a 40 per cent increase in the span taken across the diagonal of an intersection, relative to the roadway span, the magnitude of the displacements in the roof increased by a factor of four.

The other factor linked to, and affected by an increase in span, is the height to which the openings migrate in the roof, i.e. the stable roof elevation. The intersections were again compared to the roadways with the differences being converted to percentages. These changes were relatively small varying between – 5.0 and 33 per cent with the overall average being 13 per cent.

The areas that exhibited the highest percentage change of around 30 per cent were areas one and three at Colliery 'D'. Both areas were supported by full column resin bolts with an effective bolt length of 1.45 m. Although, in percentage terms, the changes appear relatively large the average stable elevations were the lowest recorded of the five areas where roadway and intersection comparisons could be made. Viewing the reactions of the five areas as a whole, the 40 per cent increase in span from the roadway width to the intersection diagonal had very little effect on the stable roof elevation. There was no evidence of a dramatic increase in the stable elevations.

***Table 3-2 Total relaxation and stable roof elevation averages***

| Colliery and Roof strata | Mining method | Support type | Monitoring Position | Total relaxation averages (mm) | Intersection percentage increase (%) | Stable roof elevation averages (mm) | Intersection percentage Change (%) |
|---|---|---|---|---|---|---|---|
| **A** Shale | C M | Full column resin | **Roadway** | 1.3 | | 0.6 | |
| **B** Coal roof 0.3 m shale then coal above | D&B | Partial column resin | **Roadway** | 2.0 | | 2.1 | |
| | | | ***Intersection*** | 5.0 | ***150*** | *2.0* | ***-5*** |
| **C** 0.3m coal with shale above | D&B | Mechanical end anchored | **Roadway** | 1.0 | | 2.2 | |
| | | | ***Intersection*** | 2.3 | ***130*** | *2.1* | ***-5*** |
| **D** Inter laminated sandstone and shale | D&B Area 1 | Full column resin | **Roadway** | 0.8 | | 1.3 | |
| | | | ***Intersection*** | 4.5 | ***460*** | *1.7* | ***30*** |
| | C M Area 2 | Partial column resin | **Roadway** | 2.5 | | 2.0 | |
| | | | ***Intersection*** | 8.0 | ***220*** | *2.2* | ***10*** |
| | C M Area 3 | Full column resin | **Roadway** | 0.8 | | 0.9 | |
| | | | ***Intersection*** | 1.0 | ***25*** | 1.2 | ***33*** |
| **E** Sandstone | C M | Full column resin | **Roadway** | 0.2 | | 0.1 | |
| **Overall average percentage change** | | | | | **197** | | **13** |

### 3.9.1 Time effects of a static face on bord stability

Although each site was visited as often as possible particularly immediately after the instrumentation was installed, it was not always possible to make direct comparisons between a large portion of the sites. The main reasons include different face advance rates for drill and blast and continuous miner sections, and the erratic nature of particular mining sequences and breakdowns. It was however possible to extract valuable information even though it may only have been recorded at a small number of sites. A typical example is the time effect on a static face.

All three examples were observed at Colliery 'D'. At area 1 site 2 (Figure 3-20), in a drill and blast section three days after the installation of the instrumentation, a second set of sonic probe and levelling readings were taken. The face had not been advanced. The results of both monitoring methods showed that the roof 0.5 m from the face was also static during this period. At area 3 site 3 (Figure 3-34) in a continuous miner section, where the face was not advanced for four days, the sonic probe readings all fell within the accepted accuracy band indicating static roof conditions.

In area 2 at site 5 (Figure 3-29) in a continuous miner section, the conditions were different in so far as the face was advanced 5.0 m to expose a dyke where it remained static over a two day period. It was then advanced through the dyke to the 11 m position and again remained stationary for three days until the final set of readings were taken. At the 5.0 m position, both the sonic probe and levelling recorded an increase in displacement of approximately 0.8 mm over the two day period. At the 11 m face position, over a three day period, no additional displacements were recorded.

These results indicate the following: close to a static face (within 0.5 m), the roof does not deform significantly. If a face remains static, the roof within its zone of influence (approximately 5.0 m away) experiences some degree of creep with time. An area of roof outside the zone of influence of the face (11 m away) is not affected by the face irrespective of whether it is stationary or advancing.

### 3.9.2 Migration mechanism

In the vast majority of cases, the final height at which the displacements in the roof stabilised was fully developed a short distance behind the face. In the drill and blast sections, the stable elevation was reached after a single blast and the face advance had increased the unsupported span to 3.0 m on average.

In the continuous miner sections, it was difficult to accurately determine at what point the stable elevation had fully developed. The reason was that half the face was usually advanced by up to 7.0 m in a single cutting sequence. After the installation of the support, the other half of the face was then advanced a similar amount before it was practical to access the sonic probe hole and take a set of readings. However, at some of the sites where the face was only advanced by 4.0 or 5.0 m (Colliery 'D' area 1, site 4; area 2, site 4, and area 2, site 5. Figure 3-22, Figure 3-28 and Figure 3-29), the stable elevations were already fully developed.

The only two monitoring sites that indicated obvious increases in the height at which displacement occurred in the roof as further mining occurred, were both in the partial column resin supported roof at Colliery 'D' (area 2 - sites 2 and 3, Figure 3-26 and Figure 3-27). Both sites were in intersections that had total relaxations amongst the highest recorded. Their total relaxation values had reached 11 mm and 5.0 mm respectively prior to the migration of the stable elevation occurring. Both the stable elevations increased quite significantly by approximately 0.5 m and 0.25 m, respectively. Since this occurred well outside the face advance zone of influence at between 56 m and 166 m and 28 m and 158 m respectively, it was time dependent behaviour.

The full displacement with time profile of a variety of elevations up to 2.5 m into the roof at site 2, area 2, Colliery 'D' are presented in Figure 3-41. Between days 11 and 59, the face was advanced from 56 m to 166 m. During this period all the strata between the 0.24 m and 2.31 m elevations in the roof deflected downwards in unison (upwards on the graph). There was no evidence of any relative displacements occurring within the 0.24 m to 2.31 m strata horizon. With time, the roof within this region deflected by about 1.0 mm allowing an additional 2.31 m elevation to become detached and deflect by a similar amount. This upward migration of the stable elevation is evident from the divergence of the 2.31 m and 2.56 m anchors in Figure 3-41.

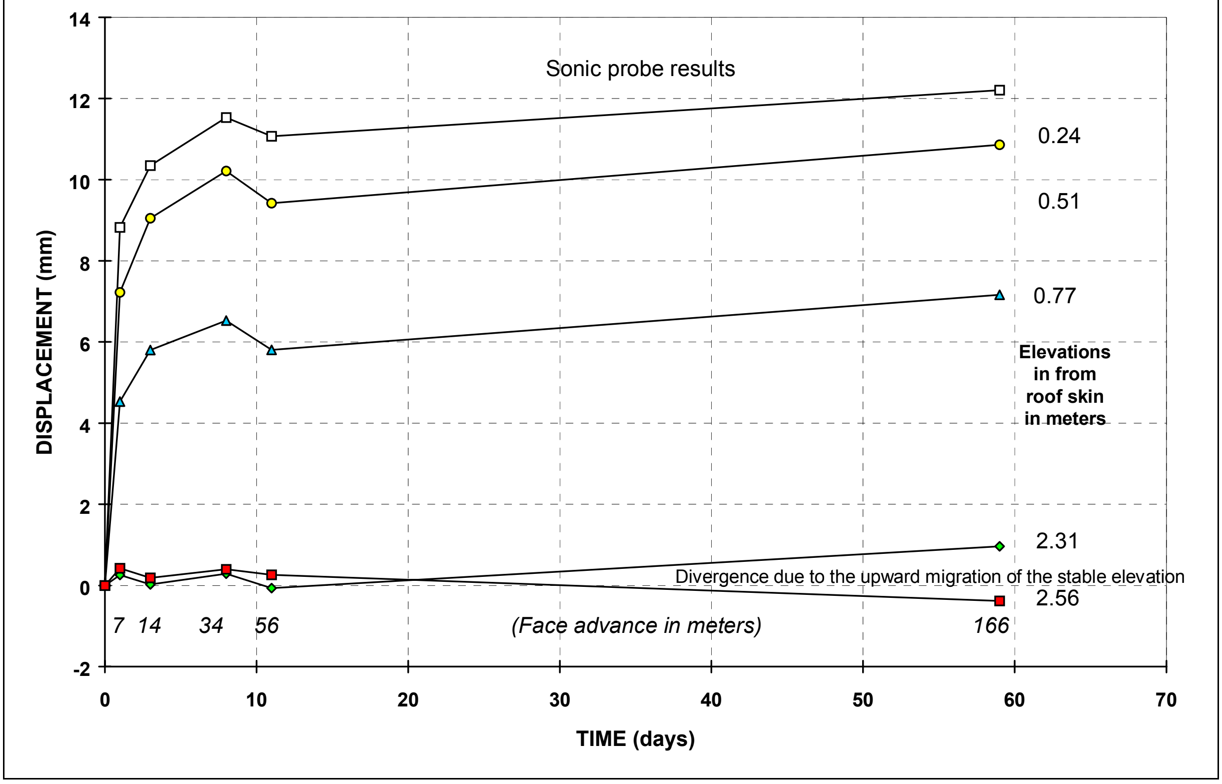


***Figure 3-41 Colliery 'D' area 2 site 2 displacements***

The roof strata at area 2, site 3, behaved in a similar manner. Two minor differences were observed. Of the approximate 1.0 mm of additional roof deflection measured close to the roof skin, about half was attributed to continued displacement within the bolt horizon. The stable elevation migration released a thinner beam that appeared to lack stiffness and came to rest on the beam below it effectively closing the parting that had existed between them as indicated in Figure 3-27. The laminated sandstone and shale roof strata at this particular colliery with its highly variable bedding thicknesses appeared to be an ideal medium for this stable elevation migration mechanism.

Awareness of this mechanism has important implications as far as roof behaviour monitoring is concerned, particularly with respect to visual indicators such as tell-tales. In the suspension support method a weaker layer of roof strata is pinned to a stiffer stronger layer above it. By positioning the top anchor point of a simple tell-tales in the stronger layer, preferably above the bolt horizon, the support performance can be monitored and remedial measures taken if it becomes necessary.

With the beam building roof support mechanism however, the choice of a suitable elevation for the top anchor point is both more complex and critical. In order to quantify a suitable elevation

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for a particular geotechnical area and support system, a roof behaviour monitoring programme should be initiated to build up a database.

Assuming that the “typical” roof strata was reasonably consistent in all three monitoring areas of Colliery ‘D’, the performance of the different support systems can be compared. The same support pattern was used in all three areas. The least effective support system was the partial resin column based support used in area 2. This was in spite of the fact that the roof, being in a continuous miner section, was not subjected to the disturbance associated with the drill and blast mining method.

In Table 3-1 the average displacement recorded in the intersections in area 2 compared to the intersections in area 1 was approximately 80 per cent larger. Even if the intersection with the headboards in area 2 is excluded, the displacement values are still on average 50 per cent larger. In addition, the displacements in the single roadway monitored in area 2 were three times the average value recorded in the two roadways in area 1.

By averaging the displacements, single curves for the intersections and roadways in each area of Colliery ‘D’ were produced and are presented in Figure 3-42. The support that performed best overall was the full column resin bolts in the continuous miner section (area 3). The apparent creep with time exhibited in the area 3 roadway curve is a function of the averaging process. There is no evidence of creep occurring at individual sites. The only curve that indicates time dependent creep is the one representing the already discussed intersections in the continuous miner partial resin column supported area 2.

Included in Figure 3-42 is the average height of the stable elevation with respect to each curve. The link between the height to which the stable elevation is restricted and the effectiveness of the support in maintaining the integrity of the roof strata within the bolt horizon is apparent in the continuous miner sections. The full column resin support in the drill and blast section however appears to be slightly out of phase with the continuous miner sections. The reasons for this are unclear but could be related to the effects that the pre-existing openings in the roof strata had on the monitoring results.

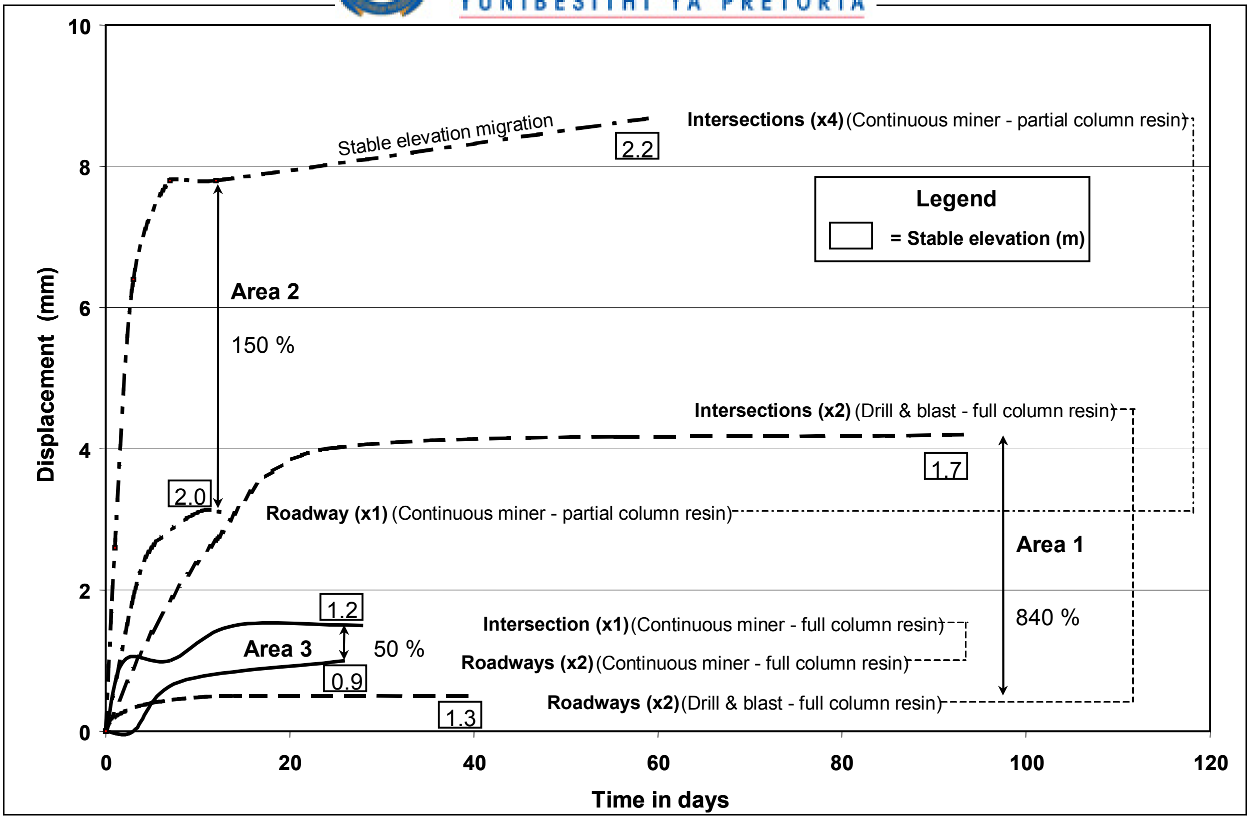


***Figure 3-42 Mining method and comparative support performance at Colliery 'D'***

## 3.10 Roadway widening

A site was located at Colliery 'A' where a section of roadway was widened from 5.1 m to approximately 12 m.

As previously mentioned, the roof in some areas of this colliery has damaged, in the form of guttering, appears to be random in nature. In the area selected for the roadway widening experiment, there was no guttering or any other obvious evidence of high horizontal stress.

The immediate roof was thick, competent sandstone unlike the roof within a couple of meters from the experiment site. Bearing in mind that in order for failure to take place the stress acting on a material should be greater than the strength of it. Therefore, although the stress was probably (not measured) same as anywhere else on the mine, because of this competent sandstone the stress damage may have not been seen in the area.

The proposed site was an existing cubby used as a waiting place. The cubby and adjoining roadways were carefully examined. No significant geological features were observed that could adversely affect the roof stability in the immediate area. The roof was supported using 20 mm diameter, 1.8 m long full column resin bolts, four bolts in a row with the rows 2.0 m apart. The mining operation was carried out by a continuous miner.

### 3.10.1 Instrumentation

The cubby was 5.1 m wide and 8.0 m long. Two sets of instrumentation were installed in the roof approximately 1.0 m from the face. Each set consisted of a 7.3 m deep sonic probe extensometer and three fixed levelling points anchored at 2.7 m, 1.8 m and close to the roof skin. Two tell-tales were also installed at each position to monitor the strata between the roof skin and the 1.8 m and 2.7 m elevations. The instrumentation layout is shown in Figure 3-43. One set of instrumentation was positioned on the centreline of the 5.1 m wide original excavation. The other approximately 1.0 m from the right hand sidewall so that it would in time be closer to the final centreline of the widened roadway. Prior to the start of the experiment, the final roadway width was unknown. The roof and sidewall conditions could only be assessed during the widening operation and a decision taken when to stop.

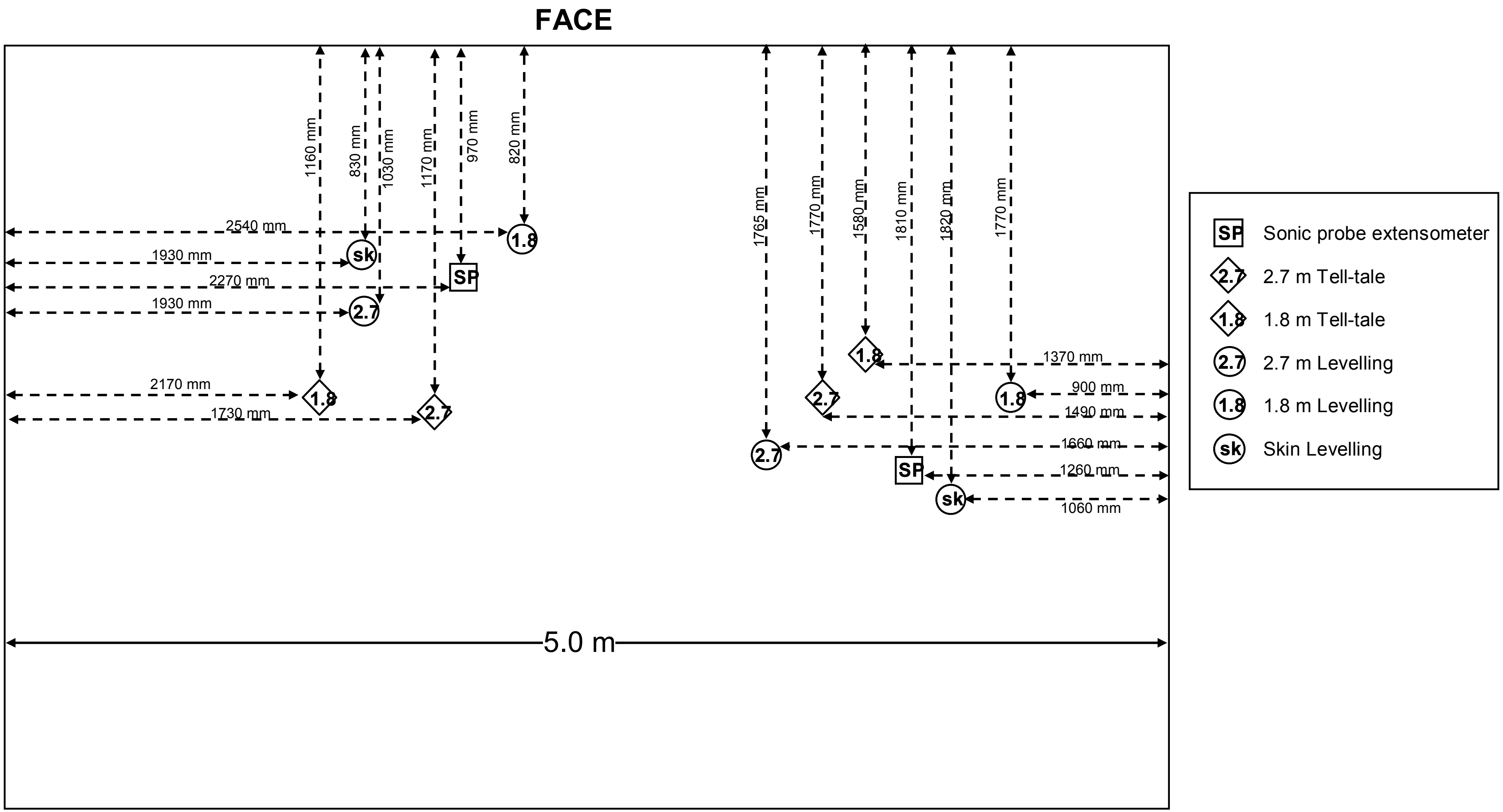


***Figure 3-43 Instrumentation layout***

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The purpose of the sonic probe installations was to gather detailed information of the roof behaviour as the cubby face was advanced and the roadway formed (in a similar manner to the other 29 sites investigated). It was also anticipated that some additional readings would be taken as the roadway widening commenced and for as long as it was safe to enter the area, if temporary support was installed. However, both sonic probe installations were damaged and were abandoned after the initial roadway was formed and the face advanced away from the site.

When the face was advanced, very small displacements were recorded close to the roof skin at both the sonic probe hole locations. The total value at the side hole was 1.0 mm while 2.0 mm was recorded at the centreline hole.

The fixed levelling points were installed primarily to be able to continue to monitor the roof remotely, during and after the roadway widening operation. To accomplish this requirement permanent levelling staves were attached to the individual fixed points protruding from the holes in the roof, immediately prior to the first widening cut with the continuous miner. These staves remained in position for the duration of the monitoring period. Four tell-tales were also installed to monitor the same sections of roof as the fixed levelling points.

A month after the initial installation, one of the fixed levelling points and most of the tell-tales were found to have been damaged. This appeared to have been as a result of the tramming of loaded shuttle cars through the site, which had a relatively low mining height of approximately 2.1 m. As a result, both the 2.7 m levelling point and telltale at the original roadway centreline site were irreparably damaged and were abandoned.

### 3.10.2 Widening procedure

The outline and dimensions of the original cubby, as well as those of the subsequent development in the immediate area, are indicated in Figure 3-44. Included are the positions of the two sets of instrumentation.

The roadway was widened in three stages as illustrated in Figure 3-45. Because of the element of risk involved, the first two cuts were stopped slightly short of breaking through into the roadway perpendicular to the one being widened. The intention was to use the behaviour of the slender pillar formed to assist in assessing the overall general stability of the area as widening of the roadway continued. Cut three was planned so as to get the sidewall as close as possible parallel to the centreline of the roadway. Based on the lack of load induced spalling on the

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slender pillar the third cut was extended until it just broke through into the roadway. This reduced the slender pillar, to becoming a snook estimated to be 0.8 to 0.9 m wide and 3.0 m long. Although it did not appear to be carrying any excessive load at the time, the risk involved in removing it was considered too high and it was left intact.

During the widening process no additional roof support was installed. At the 12 m final width, only the initial 40 per cent of the span had been supported with roof bolts.

Taking the surrounding pillars as the boundaries, the span of the final excavation was 12 m x 25 m. Even if the snook is considered as being an effective support element and boundary, the minimum dimensions are reduced to 12 m x 19 m. The monitoring was limited to the supported 40 per cent of the final roadway width. The opposite side of the widened roadway was unsupported and in all probability would have experienced larger differential displacements. Nevertheless the roof remained intact without even any minor falls being noted.

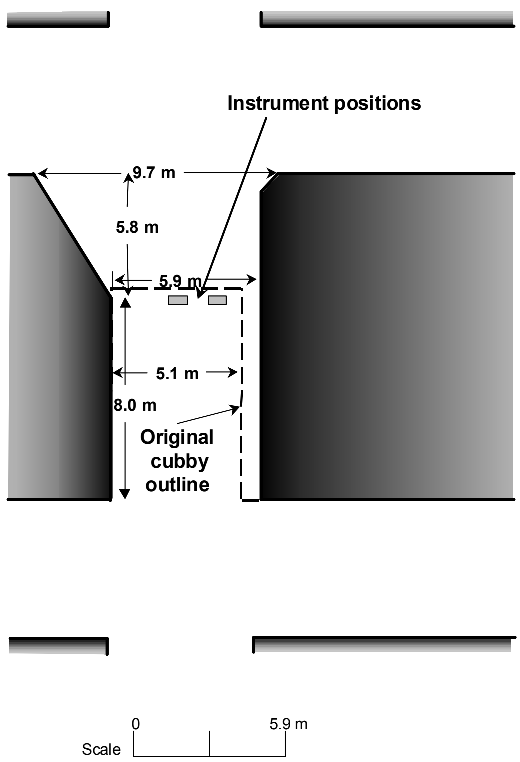


***Figure 3-44 Roadway and adjacent intersections prior to widening***

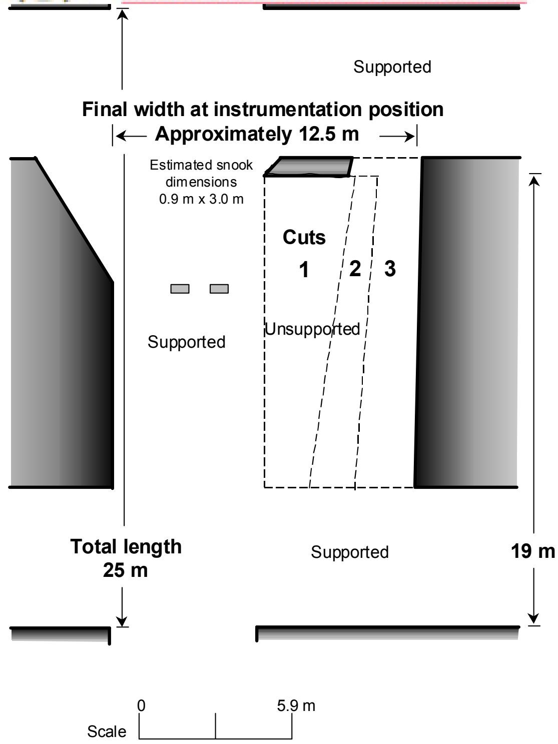


*Figure 3-45 Cutting sequence and final roadway shape*

### 3.10.3 Results

The monitoring of the roof deflection has been divided into two sections. The first is the short term dynamic performance during the widening process and the second, the longer term behaviour of the 12 m wide roadway with time. With the exception of Figure 3-48 the roof deflection and differential displacements have been given negative values.

The roof deflection recorded during the widening phase is presented in Figure 3-46. On the horizontal scale the left hand sidewall is fixed at zero, while the position of the right hand sidewall is indicated for each set of readings taken. The positions of the five levelling points are also plotted relative to the left hand sidewall. In this dynamic situation it was difficult to determine if any differential displacements, such as bed separation, were occurring. The change in shape and increase in displacement values towards the mobile centreline of the roadway occurred in the anticipated sequence in all three deflection profiles. Although not conclusive, this suggests that during this period, the 2.7 m thick roof beam being monitored remained intact.

Figure 3-47 covers the time period from day one, when the first set of readings were taken in the road once the width had been opened to 12 m, up until the final reading on day 38. The horizontal axis indicates the position of the measuring stations relative to the left hand static sidewall. In this graph two different mechanisms can be seen. The 2.7 m levelling point, being

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the deepest in the roof, traditionally gives the best indication of the overall roof deflection and is the least likely to be influenced by the unravelling effects of delamination, which usually starts close to the roof skin. The deflection of this point can be seen to increase with time. The change in shape of the five point levelling profile indicates that differential displacements were also occurring in the roof beam during this time period. These can be seen more clearly in Figure 3-48. In this Figure, the relative displacement of both the 1.8 m and skin anchor points were plotted using the 2.7 m levelling point as a static reference. The separation between the skin and 1.8 m elevation on the right hand side had started within 14 hours of the roadway reaching the 12 m width. The same comparison at the centreline showed no evidence of differential displacement at this time. This could either be as a result of an opening, once initiated, migrating towards the edge of the roadway or that the displacements were localized and not interconnected. By day nine the displacements at both instrumentation positions were well defined and tended to stabilize from day 24 onwards. The maximum differential displacements values recorded were 1.0 mm at the right hand position and 2.0 mm at the centreline. The fact that the greater value occurred closer to the sidewall suggests that these displacements may have been more localized than continuous across the roof beam.

A point worth noting, which is apparent in Figure 3-46, Figure 3-47 and Figure 3-48, is the behaviour of the 1.8 m centreline levelling point. It appears to have undergone less overall displacement than the 2.7 m point, which is anchored higher in the roof strata. This is a true reflection of the situation, which is as a result of the widening of the roadway, the 2.7 m point ended up closer to the centreline of the widened roadway. The roof deflection over the larger span influenced the 2.7 m point to a higher degree than the 1.8 m point closer to the sidewall. This was also the case with the skin anchor situated even closer to the sidewall, up until day two, when it also still recorded less deflection than the 2.7 m point.

Figure 3-49, shows the behaviour of all five points with time. After starting with relatively rapid displacement, as a result of the roadway widening process, up until day two, they continued at a fairly constant rate until day 24. From that point until when the final readings were taken on day 38, the velocity dropped 75 per cent on average.

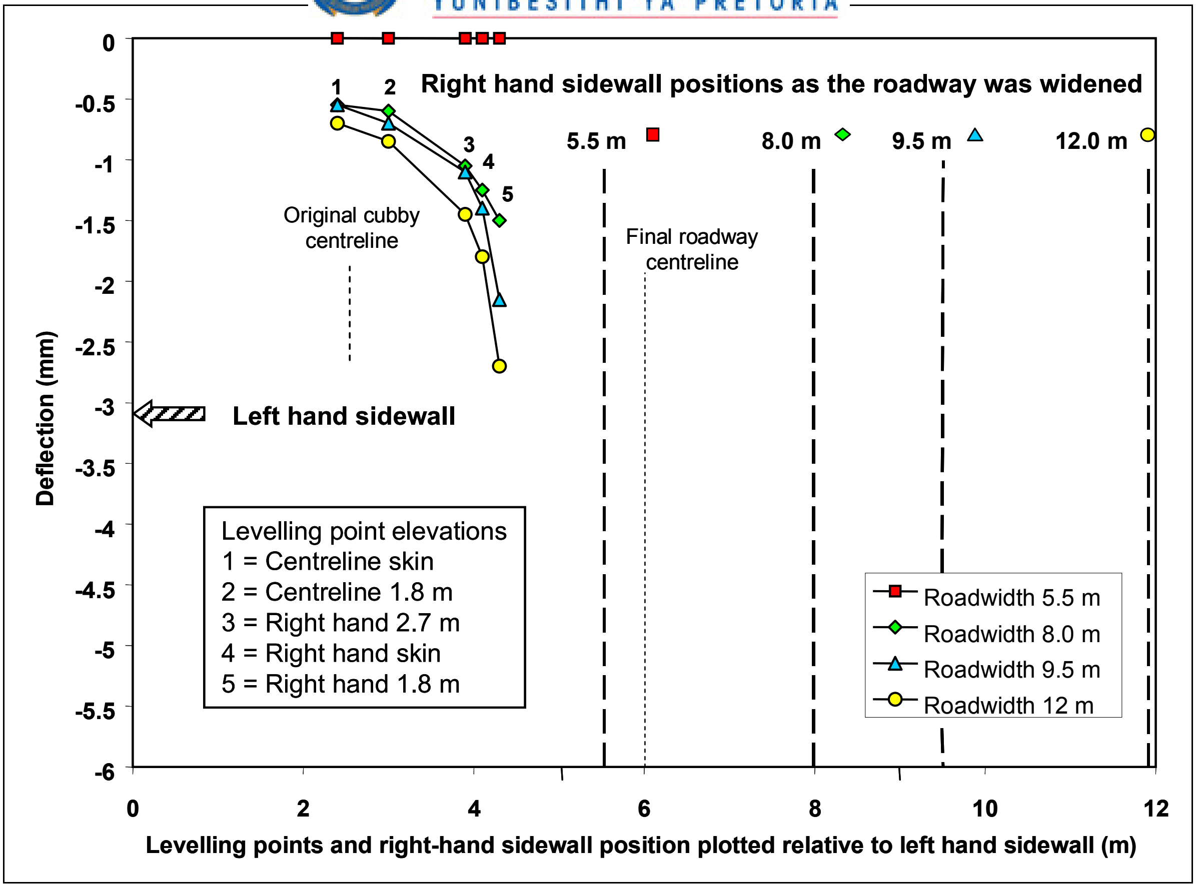


*Figure 3-46 Increase in roof deflection with widening of roadway*

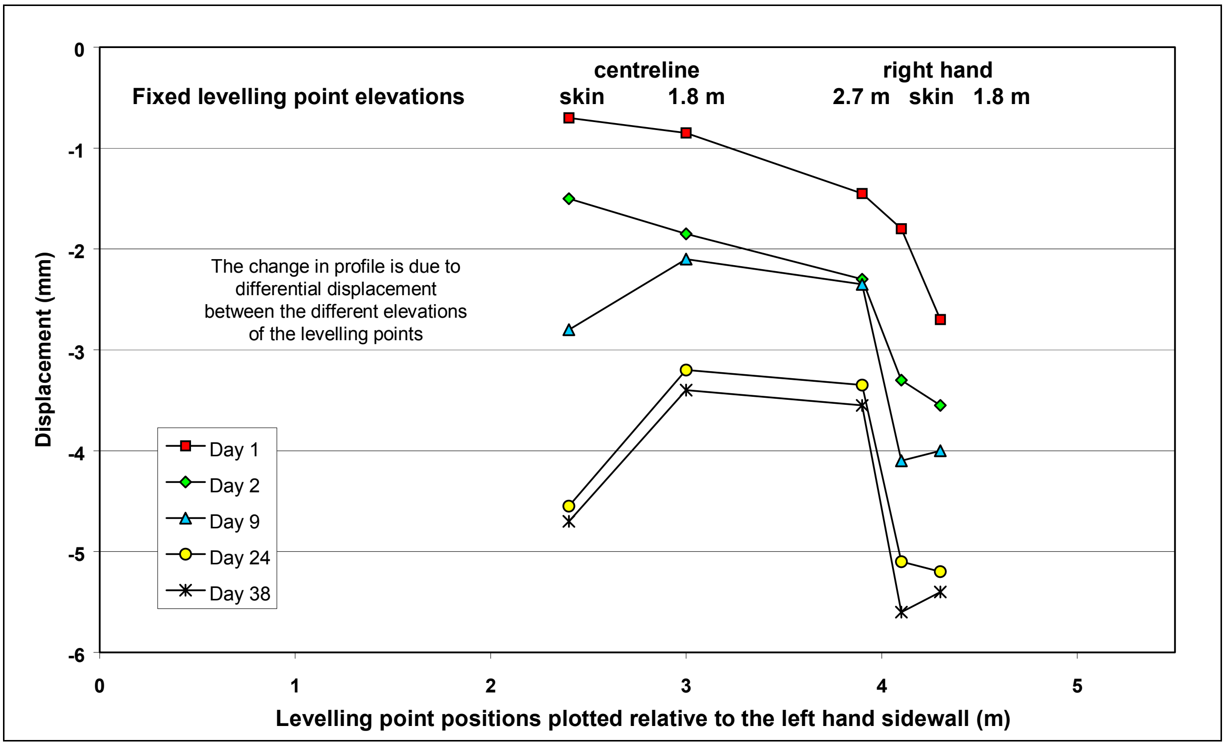


*Figure 3-47 Roof behaviour of the 12 m widened roadway with time*

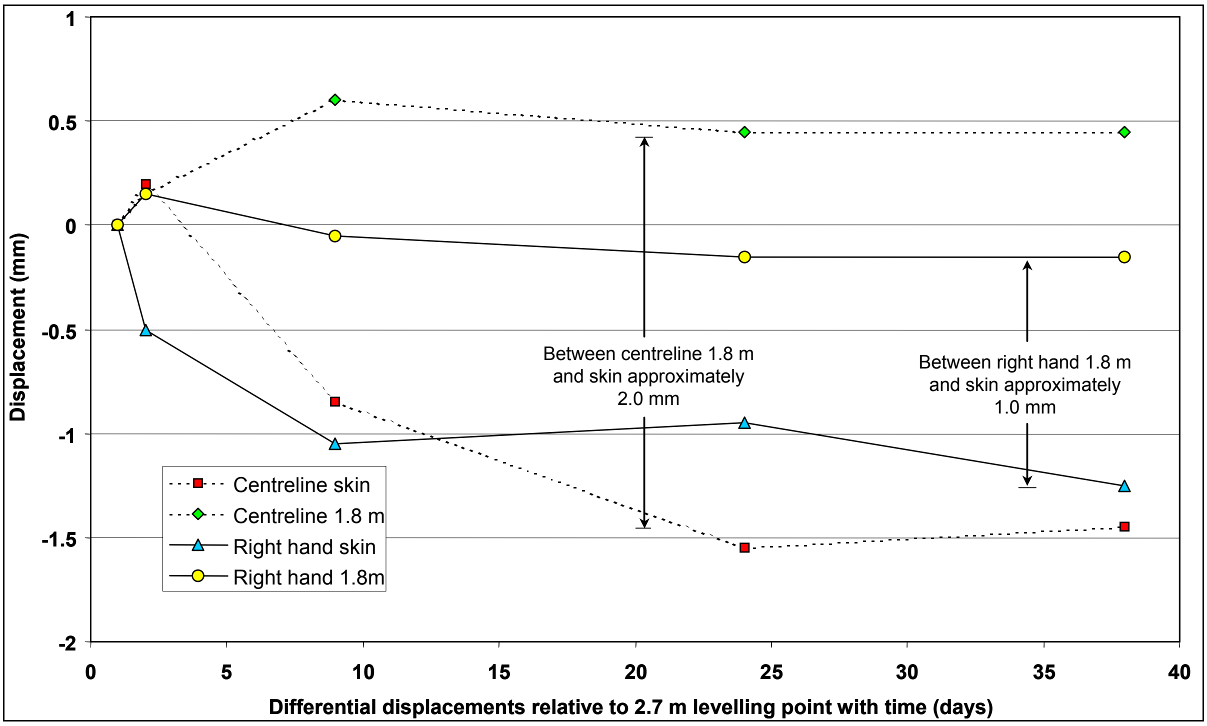


*Figure 3-48 Separation within the roof beam with time*

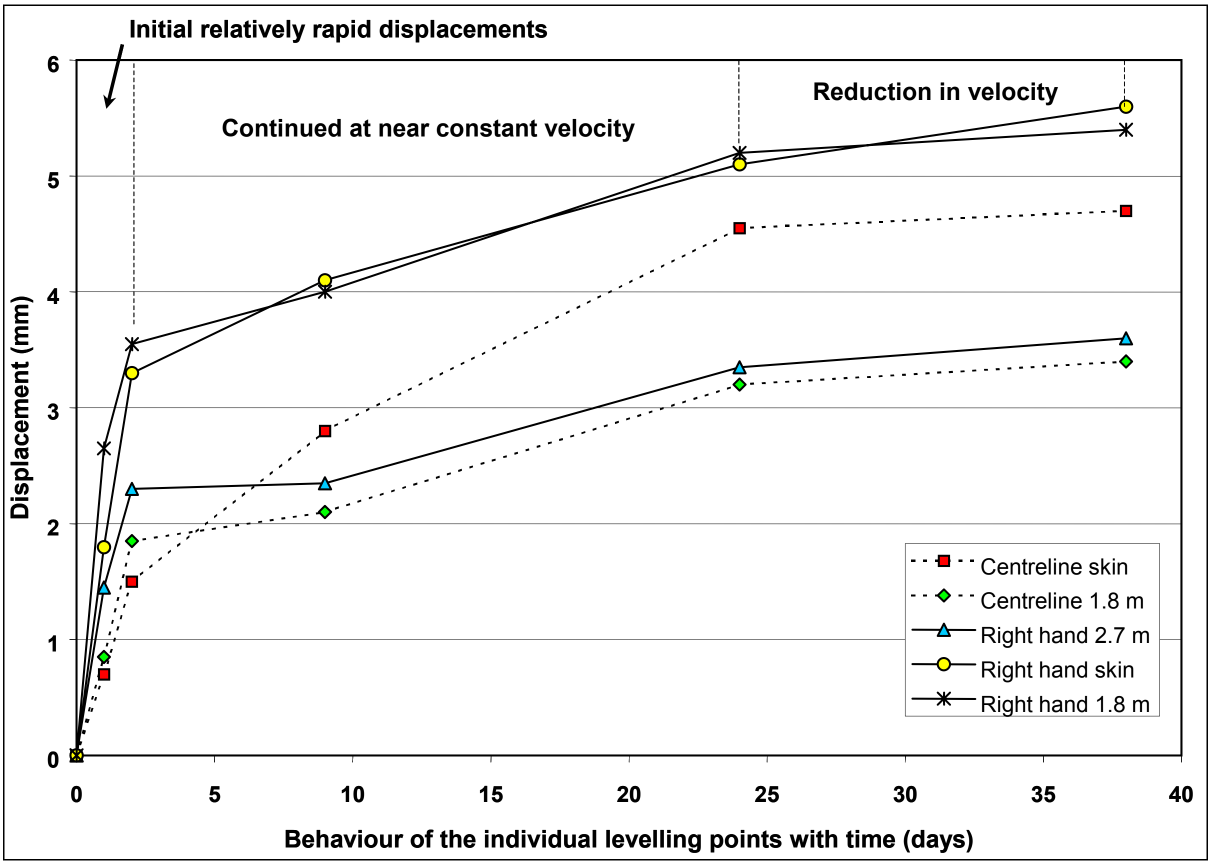


*Figure 3-49 Displacement rates as a function of time*

The photograph shown in Figure 3-50 was taken on day nine. The general stability of the area is evident from the intact corners of the snook and the clean cut edges of the break through hole to the right of it. The five permanent levelling staves and the three tell-tales can be seen in the original supported roadway on the left hand side.

Because of the distance involved, the tell-tales were observed using the telescope in the survey level. Estimates of the indicated displacements were made relative to a 5.0 mm graduated scale on each tell-tales. These were then compared to the appropriate levelling results once they had been calculated. Even though the differential levelling results were very small, the largest being 2.0 mm, they were evident and the estimated values were close to the values derived from the levelling.

The last visit was made to the site on day 52 as the area was about to be sealed off. Unfortunately all five permanent staves were missing so it was not possible to get a final set of measurements. The tell-tales were however still in position and there was no evidence of any even minor roof falls in the area. The snook was intact and appeared to have changed very little, if at all since the photograph shown in Figure 3-50 was taken on day nine.

From the tell-tales observations, the differential displacements between the two roof skin and the 1.8 m elevation had not changed and were much the same as it had been since day 24.

### 3.10.4 Conclusions

This experiment highlighted that the variations that can occur in a single mining area. Other parts of the section exhibited guttering and horizontal stress driven roof failures in supported roadways as narrow as 5.0 m.

The fact that the personnel intimately involved in the underground conditions at this particular colliery were able to identify this area as being unlikely to be influenced by high horizontal stress is significant. The signs they use to assess the presence of relatively high horizontal stress levels were absent in this particular area.

The ultimate aim of this experiment was to establish the critical roof deformations prior to roof failures. However, due to competent nature of the roof, it could not be established.

The horizontal stress driven buckling, or shearing effect, compounds the displacements induced into the roof strata by a purely gravitational loading system. This is well documented in some

Australian collieries where it is sometimes particularly severe with roof skin displacements of hundreds of millimetres being initiated as high as 5.0 m into the roof. In South Africa the cause and effects are far less dramatic as a result of which they are often not recognised as being present or taken into account in the support design procedure.

During the monitoring period no roof falls occurred at any of the 29 sites, even where 12 mm displacements were measured. As a result it was not possible to try and establish critical roof displacement values for any of the geological regions.

***Figure 3-50 Experiment site taken on day nine***

## 3.11 Conclusions

The sonic probe extensometer, which was found to be the most accurate and reliable instrument capable of monitoring roof behaviour up to 7.2 m into roof, was used throughout the underground monitoring programme. To process the monitoring data as quickly and efficiently as possibly, a customised program was written, culminating in an easy to understand set of graphic results.

A preliminary study into the height to which the openings migrate in the roof (height of roof softening), i.e. height to which instabilities could occur was conducted. In all monitoring sites all the displacements measured in the roof were confined to within 2.5 m of the roof skin. The height of instability in the intersections was compared to that in the roadways with the elevation differences being converted to percentages. These differences were relatively small, varying between -5.0 and 33 per cent with an overall average of 13 per cent.

In the vast majority of cases the stable elevation in the roof was fully developed a short distance behind the face. In the drill and blast sections, the stable elevation was reached after a single blast, where the face advance increased the unsupported span to 3.0 m on average.

In the continuous miner sections, it was difficult to accurately determine at what point the stable elevation had fully developed. The only two monitoring sites that indicated obvious increases in the height at which displacement occurred in the roof as further mining occurred, were both in the partial column resin supported roof at Colliery ‘D’ (area 2 - site two). Both sites were in intersections that had total relaxations amongst the highest recorded. Their total relaxation values had reached 11 and 5.0 mm respectively prior to the migration of the stable elevation occurring. Both the stable elevations increased quite significantly by approximately 0.5 m and 0.25 m, respectively. Since this occurred well outside the face advance zone of influence, at between 56 m and 166 m and 28 m and 158 m respectively, it appeared to be time dependent behaviour.

An investigation into the time effects of a static face indicated that close to a static face (within 0.5 m), the roof does not deform significantly. If a face remains static, the roof within its zone of influence (approximately 5.0 m away) experiences some degree of creep with time. An area of roof outside the zone of influence of the face (11 m away) is not affected by the face irrespective of whether it is stationary or be advanced.

The monitoring results also showed that there was no evidence of a dramatic increase in the stable elevations as is the case in the high horizontal stress driven beam buckling mechanism experienced in overseas collieries. It is thus concluded that in the sites monitored relatively high horizontal stress played little, if any role in increasing the deformations measured.

A roadway widening experiment was carried out to establish the critical roof displacements. The maximum width attained was 12 m at which stage ± 5 mm displacement was measured. No roof falls had occurred. However, in the same panel falls had occurred at 5 m widths. Also, falls took place in some of the areas where evidence of high horizontal stress had been noted. This indicates the significant variations that occur in a single mining area.

During the monitoring period no roof falls occurred at any of the 29 sites and road widening experiment site, even where 12 mm displacements were measured. As a result it was not possible to try and establish critical roof displacement values for any of the geological regions.

In conclusion, these results showed that the roof conditions in South African collieries can be classified as gravity loaded beams.

Relatively high horizontal stresses have been reported in South African collieries; however it is believed that these areas are isolated small areas probably affected by geological features. It is therefore important to note that when the mining is approaching towards a major geological structure, relatively high horizontal stresses may be expected and necessary precautions should be taken to reduce the effects of it on the roof deformations.

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# Chapter 4.0

# Effect of cut-out distance on roof performance

## 4.1 Introduction

One of the critical parameters in mechanical miner sections is the unsupported face advance, which determines not only the stability of the initial unsupported roadway but also that of the final supported roof. Therefore, underground monitoring programme is continued in 13 monitoring sites using 26 stations with the aim of establishing the effects of unsupported cut-out distance on roof and roof bolt performances.

While increased cut-out distances can increase production significantly, the extended cutting may endanger workers by exposing them to greater risks of injury due to roof falls. The major concern regarding extended cutting is that the unsupported roof area is larger, and that the time before permanent support installation is longer.

The standard cut-out distance of 6.0 m is regulated in other major coal producing countries and it is currently set at 12 m in South Africa. However, extended cuts (longer than 12 m) have been approved by the Department of Minerals and Energy (DME) and the maximum unsupported distance is as long as 24 m in some of South African collieries.

However, it was found that there are relatively few published references on determining effective cut-out distances as compared to other aspects of coal mining. References covering various aspects of the problem were selected and are summarized together with regulations for extended cut-out distances in major coal producing countries.

## 4.2 Research conducted

The references relevant to cut-out distances include remote-control operation of continuous mining machines, the control of dust and methane, elimination of frictional ignitions, effective ventilation methods, and human factors (worker/machine interaction). Cut-out distance ground control aspects are also mentioned in relatively few references. This thesis investigated the ground control problems associated with extended cut mining. Therefore, only the literature which deals with ground control aspects of extended cut mining was reviewed.

Remote control, ventilation and human factors aspects of extended cut mining can be found in the following references:

| | |
|---|---|
| Remote control mining: | Warner (1973a), Lindsay (1973), Davis (1977). |
| Ventilation: | Divers et al. (1982), Taylor et al. (1992), Volkwein et al. (1985), Campbell (1979), Jayaraman (1987). |
| Human factors: | King and Frants (1977), Sanders and Kelly (1981), Love and Randolph (1991 and 1992), Randolph (1992a). |

The majority of research into the effects of extended cut-out distances on ground control have been conducted in the USA by the National Institute for Occupational Safety and Health (NIOSH) during the period 1993 to 1998.

Bauer et al. (1993) conducted a preliminary examination of coal mine roof-fall fatalities from 1988 through 1992. They reported that extended cutting was a contributing factor in approximately 23 per cent of the fatal roof falls, and that geology was an influence in over 80 per cent of the roof fall fatalities in both extended- and non-extended-cut mining. They also reported that nearly 65 per cent of the extended-cut roof fall fatalities were the result of non-approved extended cutting (mining of cuts deeper than 6.0 m without an extended-cut permit or mining deeper than the approved extended-cut depth). Overall, the fatality rate was found to be 37 per cent lower for extended-cut mines. They concluded that in nearly 40 per cent of all roof fall fatalities, the victim was behind the last row of permanent support (Bauer, 1998).

Grau and Bauer (1997) reported on an underground study that addressed the long-term stability of extended-cut areas (over a 10 month period) as compared to non-extended-cut areas. They used a rating system modified from one developed by Mucho and Mark, 1994. The long-term stability was analysed by comparing how cuts in each rating category changed and how the extended cuts changed with respect to the nonextended-cuts. They concluded that a high percentage of extended cuts experienced roof damage over time even though these areas initially had stable roof conditions. In non-extended cut areas where changes occurred, the damage was more severe.

Bauer (1998) investigated site specific stability associated with the mining of extended cuts. He concluded that there was no significant increase in roof fall incidence rates after the mines were granted approval to mine extended cuts. The underground investigations revealed a relationship between depth-of-cut and roof conditions; i.e. that extended cuts were generally mined where the roof was stable and non-extended cuts were mined where the roof showed signs of instability. Also, the study indicated that extended cuts were twice as likely to experience changing roof conditions over time than non-extended cuts. He stated that this occurred because it was easier to detect changing roof conditions in areas originally found to have no

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visible stability problems (the areas where extended cuts are mined), than it was to detect changes in areas already experiencing stability problems.

Bauer (1998) also found that there was an increase in worker injuries during the remote-control mining of extended cuts. Accident and fatality information suggested that the mining of 90 deg. crosscuts presented additional worker-safety concerns. An alternative shown to minimise these concerns was the mining of angled crosscuts instead of right- and left-hand $90^0$ crosscuts.

Two dimensional (2D) numerical modelling was also conducted to understand roof and pillar reactions during extended-cut mining. The numerical modelling successfully predicted where roof displacements would be expected to occur, and delineated the roof-stability concerns caused by geological discontinuities.

Bauer (1998) established the following formula to estimate the safe cut-out distances:

$$CutDepth = 8.1 + 0.564\,(CMRR) - 0.152\,(B) - 0.0029\,(H) \qquad [4\text{-}1]$$

where $CMRR$ = Coal Mine Roof Rating
$B$ = Bord width (ft)
$H$ = Depth below surface (ft)

Bauer (1998) also investigated the applicability of analytical solutions to determine the safe cut-out distances. He concluded that the strength of the rock is not as important in determining the maximum safe cut-out distances as is the type, number and/or frequency of discontinuities in the immediate roof. Finally, he suggested that until another method is proposed, tested, and verified, the decision as to the safe depth of each individual cut must be left to the CM operator.

Bauer (1998) stated that one of the major concerns of extended cuts is the time delay for support installation. In general, it is expected that support should be installed as soon as the mining takes place to prevent bed separation. The stand-up time is dependent upon the geotechnical parameters in the roof of the excavation. Where mobilisation of low friction parting planes occur, the beds delaminate inducing tensile and shear forces which can cause the beam to fail.

The tensile strength of rock is only 1/10 of the compressive strength, and strata failure is often initiated by tensile cracks at the edges of the unsupported span. With time these cracks grow. Also, the material is affected by oxygen and moisture (ventilation), which decrease its inherent strength with time, van der Merwe, 1995.

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Currently, the effect of time on roof behaviour cannot be quantified mathematically, although, there have been studies to identify the effect of time on support and roof performance.

Buddery (1989) suggested that, in order to gain maximum benefit, roof bolts should be installed as soon as possible after the roof has been exposed as this will limit the amount of roof deflection and bed separation. Small cut-out distances are therefore implied.

Radcliffe and Stateham (1980) investigated the interval between exposure and support of the roof, in a mine in the USA, using both instrumentation and statistical methods. Instrumentation studies were designed to equate displacement and rates of displacement with areas of roof left unsupported from 15 minutes to more than four days. A statistical study was completed to compare roof fall occurrence with time-lapse intervals encountered during normal bord and pillar mining. Results from this investigation showed the time lapse, in this specific mine, to be insignificant with respect to roof stability.

The following basic relationships that govern stand-up time were originally formulated by Austrian tunnelling engineers (Mark, 1999).

- for a given rock mass, a tunnel's stand-up time decreases as the roof span becomes wider, and
- for a given roof span, a tunnel's stand-up time decreases as the quality of the rock mass reduces.

Using data collected from numerous tunnels and mines, Bieniawski (1989) was able to quantify this relationship. Bieniawski used Rock Mass Rating (RMR) as the measure of rock quality. His data indicated that an unsupported 4.3 m wide tunnel would be expected to collapse immediately if the RMR of the roof was less than 33. If the tunnel was 6.0 m wide, immediate collapse would be expected if the RMR was less than 41. The following equation expresses the relationship for this range of tunnel spans (approximately the range encountered in underground coal mining), (Mark, 1999).

$$RMR = 13 + 1.4\,B \qquad [4\text{-}2]$$

where $B$ is the bord width, in feet.

Mark (1999) stated that because roof bolting normally takes place within several hours of mining, the collapse of an extended cut may be considered "immediate".

In order to identify the lithological factors that influence the structural competence of a mine roof, Molinda and Mark proposed a Coal Mine Roof Rating (CMRR) in 1994.

In developing the CMRR, field data were collected from nearly 100 mines in every major coalfield in the USA.

Mark (1999) used the CMRR to determine stand-up times at 36 mines with a questionnaire being used to identify the stand-up times. The results were divided into three classes: Class 1 "always stable", Class 2 "sometimes stable" and Class 3 "never stable". Mark concluded that the CMRR/depth of cover and CMRR / bord width are statistically significant to determine the stability in the extended cut sections using the CMRR.

The following relationships for CMRR-depth of cover and CMRR-bord width are given respectively by Mark (1999):

$$CMRR_{crit} = 40.9 + (H/100) \qquad [4\text{-}3]$$

$$CMRR_{crit} = 19.2 + 1.64\,B \qquad [4\text{-}4]$$

where $CMRR_{crit}$ = CMRR value below which instability may start to occur.

$H$ = depth below surface (ft)

$B$ = the road width (ft)

### 4.2.1 Summary of current knowledge

In South Africa the standard cut-out distance is 12 m. However, if the roof is defined as self-supporting then systematic support is not required and work under unsupported roof is permitted. Under these conditions defining a maximum cut-out distance becomes irrelevant with respect to ground control, and the issue becomes one of sufficient ventilation at the face and dust control.

The issues which are given most consideration in determining cut-out distances include remote-control operation of continuous mining machines, the control of dust and methane, elimination of frictional ignitions, effective ventilation methods, and human factors (worker/machine interaction). Given the general requirement that no person should be allowed under unsupported roof, roof stability is seldom considered a major issue in determining cut-out distance and few references covering this topic could be found. Nevertheless a number of detailed studies relating stand-up time to rock mass quality and mining dimensions have been carried out and could be used for determining maximum cut-out distances. A serious limitation is the possibility of unexpected changes in roof stability, and also verification of the empirical relationships would have to be carried out for local conditions.

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However, further research into the effects of extended cut-out distances on ground control was conducted in the USA by the National Institute for Occupational Safety and Health (NIOSH) during the period 1993 to 1998 by Bauer (1998). He concluded that extended cut-mining is about as safe as the mining of non-extended cuts from a roof fall accident and fatality perspective, mainly because extended cut mining was only allowed in good quality roof conditions.

## 4.3 Underground monitoring

### 4.3.1 Introduction

In order to determine the displacements in the roof for various cut-out distances, in different geotechnical areas, an underground monitoring programme was carried out. A total of 13 sites at six collieries in four seams were monitored as part of this task. Approximately 80 per cent of coal production comes from Witbank and Highveld Coalfields, and therefore the monitoring sites were concentrated in these two coalfields.

***Table 4-1 Distribution of test sites***

| Number of sites | Colliery | Sites | Seam |
|---|---|---|---|
| 1 | S | 1 | Vereeniging 2B |
| 10 | A | 4 | Witbank No 2 |
| | B | 2 | Witbank No 2 |
| | G | 1 | Witbank No 2 |
| | K | 3 | Highveld No 2 |
| 2 | T | 1 | Highveld No 2 |
| | B | 1 | Witbank No 5 |

Two sonic probe extensometers were again used to monitor the roof and support behaviour in the sites. In the initial tests it was observed that drilling a 7.0 m hole into the roof was difficult and in many sites there was not a proper drilling machine available. Therefore, a 4.0 m sonic probe extensometer was used as drilling of this length hole was more readily accomplished.

### 4.3.2 Underground monitoring procedure

In order to monitor the roof behaviour in continuous miner and road-header sections, two different monitoring programmes were established, which suited the mining cycles in both mechanical miner sections.

Two different cutting sequences were used in the experiments to cater for different mining equipment.

**CM sequence**: The full cut length was completed in four steps, Figure 4-1. The first sonic probe hole was drilled and instrumented next to the last row of the support approximately 1.0 m from the face. The face was then advanced by half of the standard cut-out distance with a single drum cut and the second hole was drilled and instrumented at the face. Then, the second, third and fourth lifts were cut. A sonic probe reading was taken following the first, second and the fourth mining steps to monitor movements into the roof as mining takes place.

**Road-header sequence**: The full cut length was completed in two steps, Figure 4-2. The first sonic probe hole was drilled and instrumented just behind the last row of support, approximately 1.0 m from the face. The face was then advanced by half of the standard cut-out distance in full bord width. The second monitoring hole was drilled and instrumented at the face. Then the second half of the full length was cut. A sonic probe reading was taken after each mining step.

To record all the information relevant to roof strata deformation prior to the installation, the first monitoring holes were drilled and instrumented close to the last row of support, and the second hole approximately 1.0 m away from the face in the unsupported ground.

Drill bit sizes, resin quantities and support types and lengths were recorded in each monitoring site. This information is presented in each graph from each monitoring site. In addition, the support installation and drilling were also monitored in each section. However, the performance of the drilling crew tends to improve when the crew is being observed. Therefore, it was decided that the support installation should be monitored in the sections by visual observations, and van der Merwe's (1998) support installation and roof damage checklists were adopted in each site, Figure 4-3.

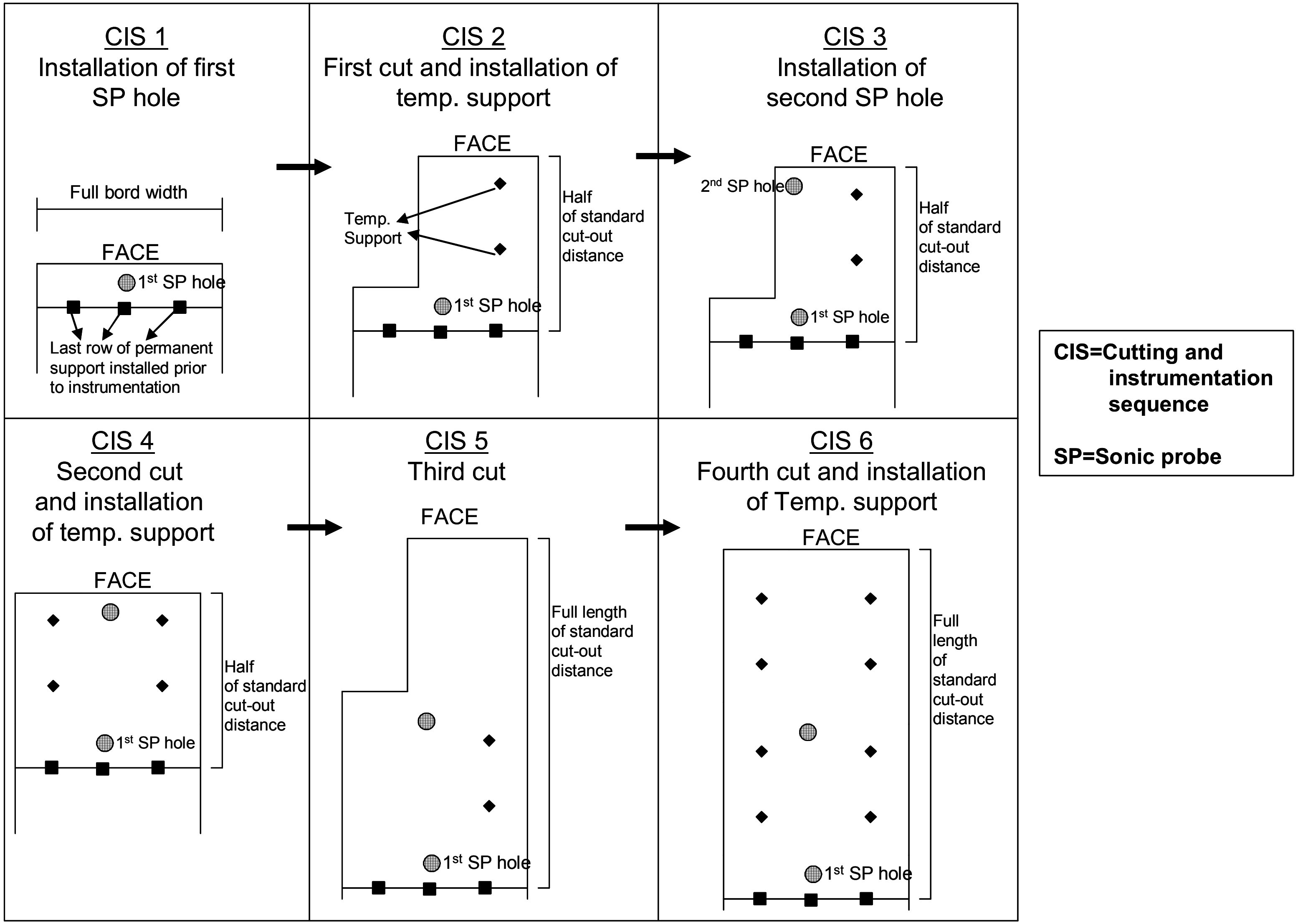
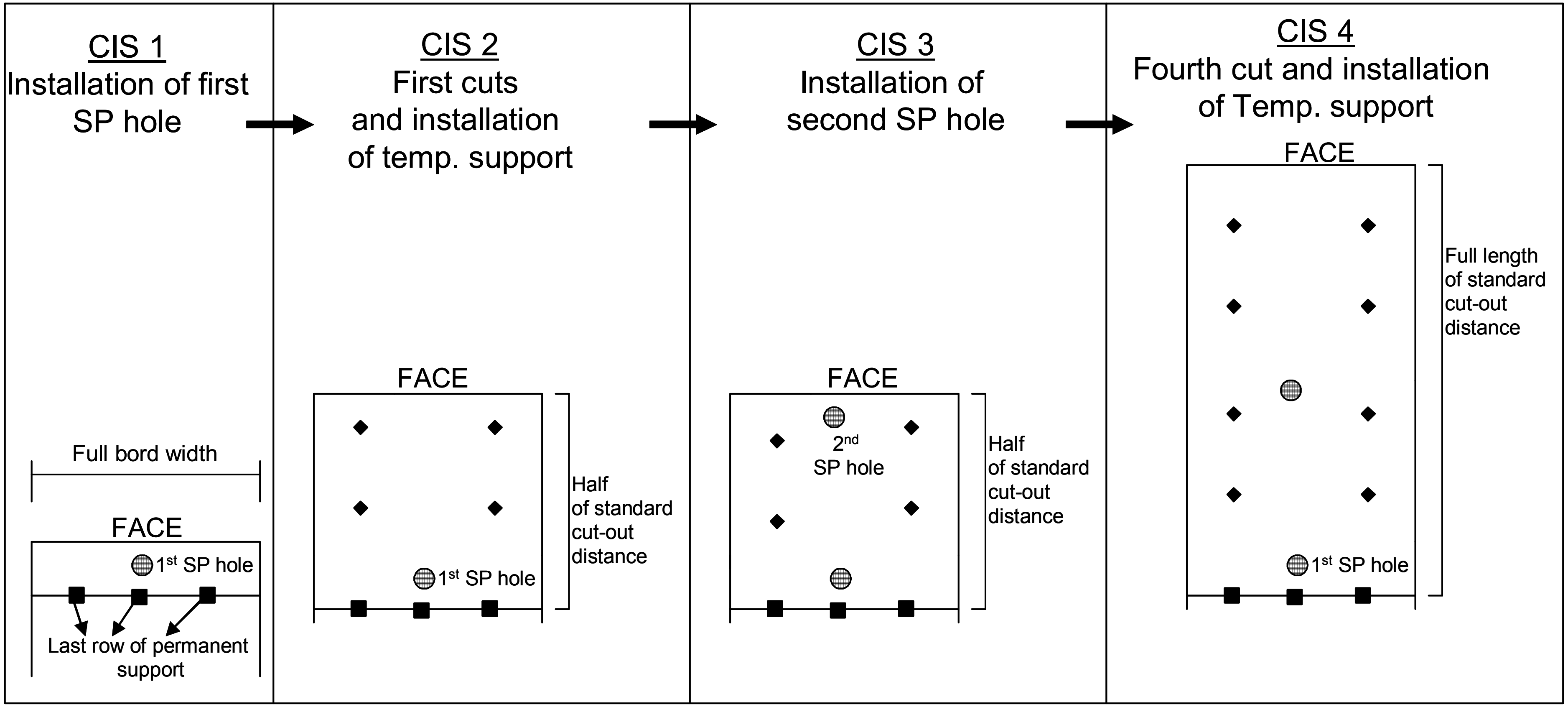



***Figure 4-1 Cutting and instrumentation sequence in CM sections***




**CIS=Cutting and instrumentation sequence**

**SP=Sonic probe**

***Figure 4-2 Cutting and instrumentation sequence in road header sections***

| Observation \ Cause | Installation problems: Hole too short | Hole too long | Weak washer | Spin/wait times wrong | Temperature | Insufficient tension | Worn adaptor | Crimp too strong | Crimp too weak | Torque too high | torque too low | System problems: Spacing too wide | High stress |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Too much thread protruding | ● | | | ◑ ⊕ | ◑ ⊕ | | | | ○ | ○ | | | |
| Too little thread protruding | | ● | | | | | | ◑ ⊕ | | | ○ | | |
| Loose washer | ◑ | | | ○ ⊕ | ○ | ● ⊕ | | ◑ ⊕ | | | ○ | | |
| Rounded nut | | | | | | | ● ⊕ | ○ | | ◑ ⊕ | | | |
| Deformed washer | | | ◑ | | | | | | | ○ | | ○ | ● ⊕ |

● Most likely cause ◑ Possible cause ○ Potential cause ⊕ Most serious

(a)

| Observation \ Cause | Roadway too wide | Spacings too wide | Bolts too short | Insufficient tension | High horizontal stress | Weak anchor | Weak steel | Support installed too late | Alarm rating |
|---|---|---|---|---|---|---|---|---|---|
| Tension crack | ● | | ◑ | ○ | | ○ | | ◑ | 1 |
| Guttering | ◑ | | | | ● | | | ◑ | 1 |
| Falls, bolts broken | ○ | ◑ | | | ○ | | ● | | 2 |
| Falls, bolts pulled out | ◑ | | ○ | | ○ | ● | | | 2 |
| Falls higher than bolt length | ◑ | | ● | | ○ | | | ○ | 2 |
| Falls, bolts hanging free | | ◑ | | ● | | ○ | | ○ | 3 |
| Falls, rock columns on top of bolts | ○ | ● | | | | | | ◑ | 3 |
| **Seriousness rating** | 1 | 3 | 4 | 6 | 4 | 5 | 7 | 2 | |

● Most likely cause ◑ Possible cause ○ Potential cause

**Seriousness rating: 1 is the most serious non conformance**

**Alarm rating: 1 is the most dangerous situation**

(b)

*Figure 4-3 a) Probable cause of observed roof damage. b) Probable cause of observed roof bolt defects (after van der Merwe, 1998)*

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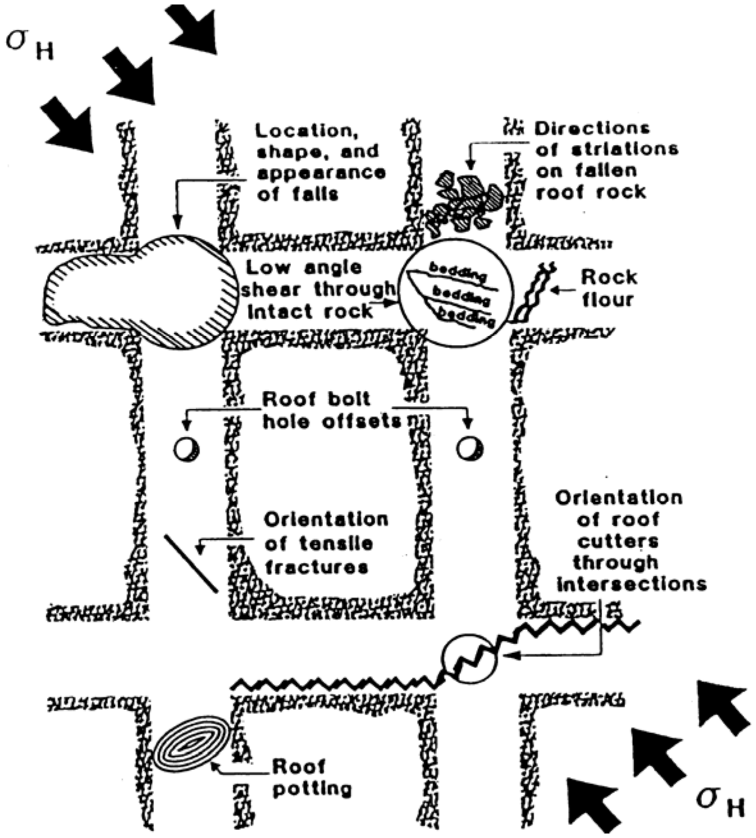
Horizontal stress manifests itself in a variety of features that can be observed underground. Therefore, indicators of horizontal stress in each section and site were also carefully monitored using the technique developed by Mark and Mucho (1994), Figure 4-4.




| Feature | Observation Noted | Relationship to $\sigma_H$ |
|---|---|---|
| CUTTER - GUTTERING OR KINK ROOF | Location in entry, especially tendency passing through intersections | Entry location gives indication of angle of mining to stress field. Through intersections tries to align with $\sigma_H$ |
| TENSILE FRACTURES | Direction | Gives direction of $\sigma_H$ |
| ROOF POTTING | Direction of major and minor axes | Major axis gives direction of $\sigma_H$ |
| ROOF BOLT HOLE OFFSETS | Direction of roof movement | Roof layers move in direction of $\sigma_H$ |
| SHEAR PLANES AND ROCK FLOUR | Direction | Planes and rock flour lines are in the direction of $\sigma_H$ |
| STRIATIONS ON ROOF ROCK | Direction | Striations are parallel to $\sigma_H$ |
| ROOF FALLS | Location , shape and appearance | Location gives clues as to the general directionality of the stress field. High angular shape usually indicates high horizontal stress with stepped shear failures usually predominating on one side |

***Figure 4-4 Summary of underground stress mapping techniques (after Mark and Mucho, 1994)***

Depending on the mining method and rate of face advance, the time lapse between further sets of readings varied from hours to days. In a typical development section underground, the centre roadway in the section (belt road) was usually monitored. When monitoring in the belt road was not possible, the holes were drilled in the closest roadway to the belt road. The reason for this was for the tensile stress and deformations to be at their maximum in the middle of the panels.

Initially, it was planned that in each site petroscope holes should be drilled next to each sonic probe hole in order to gain maximum information. However, it was found that, because these experiments interfered significantly with production, it was not possible to drill extra holes, which would further delay the production and support installation in the sections. Also, the possibility of underground core drilling to obtain the stratigraphy of the first 2.0 m into the roof was investigated. Difficulties were experienced in drilling with the available roofbolters and problems also arose because of delays caused to production. Therefore, it was decided not to core drill in the sections, where experiments took place. However, detailed borehole logs from the vicinity of the experiment sites were obtained from the geology departments at each colliery. The detailed logging of the immediate roof strata from those boreholes is also presented in each graph from each monitoring sites.

### 4.3.3 Processing of information

The information obtained from each site was processed similar to previous chapter. However, modifications are made in the presenting the results; the results from both sonic probe extensometer holes are presented in one figure in which the individual graphs from each hole have been cropped at the 3.0 m elevation.

After the installations were completed, the initial readings were taken from both holes. These comprise a minimum of three sets from each hole, which were screened for any obvious anomalies or booking errors. They were then entered into the program where they were averaged, and the calculations carried out to produce the graphic results necessary for interpretation. All the subsequent sets of readings were treated in a similar manner with the program comparing them to the first (datum) set of readings from which the displacements were calculated.

## 4.4 Colliery 'A'

Four sites in three different sections were monitored at Colliery 'A'. The colliery is situated in the Witbank Coalfield and mining No 2B Seam at a depth of 32 to 59 m using a continuous miner. While the CM experiment sequence was used in the first two sites, the road header sequence was used in the remaining two sites.

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### 4.4.1 Colliery 'A' Site 1, Test 1

In Site 1 two experiments were conducted approximately 150 m apart from each other. Site 1 was an eight-roadway, primary bord and pillar production section, and in both experiments the sonic probe monitoring holes were drilled in one-left roadway (one left-hand from the centre, belt road). Because of a major water aquifer 5.0 to 6.0 m into the roof, some degree of damage in the workings was observed. Initially, it was aimed to drill 8.0 m sonic probe holes into the roof, however, because of the aquifer, the holes were limited to 5.0 m into the roof. Also, scaling in the pillar - roof contacts indicated some degree of horizontal stress. This was confirmed by the stress mapping technique. Installation and performance of support were found to be excellent in the section.

After the installation of the first hole was completed, the initial reading was taken from this hole. Then the face was advanced by 8.0 m in full bord width of 5.8 m, and the second hole drilled and instrumented. Readings were taken from both holes. Then, the face was advanced a further 8.0 m and readings were taken from both holes. The face was left unsupported for 48-hours in order to determine the effect of time on deformation. After 48-hours readings were taken and entered into the program. Further readings were taken 5 and 11 days after the support installation. The face advance was approximately 30 m, when the last reading was taken.

The results obtained from both holes during the first experiment in Site 1 are presented in Figure 4-5. The summary of site performance in this site is given in Table 4-2.

***Table 4-2 Site performance Colliery 'A' Site 1, Test 1***

| | | | |
|---|---|---|---|
| Coalfield: | Witbank | Seam: | No 2 |
| Site: | One – Test 1 | Positions: | Roadway |
| Road widths: | 5.8 m | Pillar widths: | 9.0 m |
| Mining height: | 3.0 m | Depth: | 32 m |

| | |
|---|---|
| Mining method: | CM and shuttle cars |
| Cut-out distance | 16 m |

| | |
|---|---|
| Roof strata: | 0.13 m grit, 0.65 m grit/coal, and 0.08 m coal overlain by 0.74 m thick sandstone |

| | |
|---|---|
| Support: | 1.5 m x 20 mm OZ-Bar, full column resin in 25 mm hole.<br>Support density 0.57 bolt/m$^2$. |

| | |
|---|---|
| Performance: | Although there were indications of the presence of high horizontal stress within the section, there was no visual evidence to indicate its development at either monitoring site.<br><br>Roof separation was only measured at one site up to a maximum of 1.1 m into the shale allowing the roof skin a total relaxation of 2.5 mm after a face advance of 66 m over a 66 day period. |

### 4.4.2 Colliery 'A' Site 1, Test 2

Because the experiment sites in Site 1 were very close to each other, descriptive information obtained in the first experiment was used for the second experiment.

The same cutting and instrumentation sequence as Test 1 was used during the second experiment. The results obtained from both holes during the second experiment in Site 1 are presented in Figure 4-6. In this experiment the final readings were taken after 542 m face advance, which indicated that even after this face advance, the displacement in the roof was not significant (± 0.5 mm). The summary of the site performance in this site is given in Table 4-3.

***Table 4-3 Site performance Colliery 'A' Site 1, Test 2***

| | | | |
|---|---|---|---|
| Coalfield: | Witbank | Seam: | No 2 |
| Site: | One – Test 2 | Positions: | Roadway |
| Road widths: | 5.8 m | Pillar widths: | 9.0 m |
| Mining height: | 3.0 m | Depth: | 32 m |
| Mining method: | CM and shuttle cars | | |
| Cut-out distance | 16 m | | |
| Roof strata: | 0.13 m grit, 0.65 m grit/coal, and 0.08 m coal overlain by 0.74 m thick sandstone | | |
| Support: | 1.5 m x 20 mm OZ-Bar, full column resin in 25 mm hole.<br>Support density 0.57 bolt/$m^2$. | | |
| Performance: | Approximately 1.5 mm dilation, 1.5 m into the roof, was observed in No 1 hole. Initial dilation of 1.0 mm was recorded after the completion of 16 m | | |

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unsupported face advance. No further dilation was recorded after 48-hours stand-up time. Further 0.5 mm dilation took place after the face advanced by 542 m.

The total dilation in the second hole, which took place 1.0 m into the roof at the same interface of grit/coal

### 4.4.3 Colliery 'A' Site 2

Site 2 was an 11-roadway primary bord and pillar production section, and the sonic probe monitoring holes were drilled in the two-left roadway. Similarly, in some localised areas in the section, floor heave and scaling of roof-pillar contact indicated horizontal stress driven damage. The underground dimension control, installation and performance of support were excellent in the section.

After the installation of the first hole was completed, the initial reading was taken from this hole. The face was then advanced by 8.7 m at full bord width and the second hole drilled and instrumented with sonic probe anchors, and readings were taken from both holes. Advancing the face by 8.0 m, the full cut-out length of 16.7 m was completed. The readings were again taken from both holes. The face was left for 48-hours to monitor the effect of stand-up time. Further readings from both holes were taken 96 hours after the support installation, 15 days after the support installation (approximately 50 m face advance), and after 364 m face advance.

The results obtained from both holes during the experiment in Site 2 are presented in Figure 4-7. The results showed that after 364 m face advance, No 2 hole indicated some degree of displacement. However, it is known that after the initial phase of the experiment (up to 15 days after support installation, as indicated in Figure 4-7), an intersection was developed between the two holes during the mining cycle. Therefore, it was thought this movement was due to stress changes in the area. The summary of the site performance in this site is given in Table 4-4.

***Table 4-4 Site performance Colliery 'A' Site 2***

| | | | |
|---|---|---|---|
| Coalfield: | Witbank | Seam: | No 2 |
| Site: | Two | Positions: | Roadway |
| Road widths: | 6.85 m | Pillar widths: | 7.5 m |
| Mining height: | 3.4 m | Depth: | 59 m |
| Mining method: | Road header and shuttle cars | | |

Cut-out distance 16.7 m

Roof strata: 0.54 m sandstone overlain by 1.8 m shale

Support: 0.9 m x 20 mm OZ-Bar, full column resin in 25 mm hole.
Support density 0.41 bolt/m$^2$.

Performance: No 1 hole was stable throughout the experiment period.

The total dilation recorded in the No 2 hole was 1.5 mm, which took place 0.5 m into the roof at the sandstone shale contact. This displacement took place between 60 m to 364 m face advance. Initially, this movement was thought to be due to the effect of face advance. However, detailed investigation showed that an intersection was developed during this period, and this movement was due to stress changes during the development of the intersection.

#### 4.4.4 Colliery 'A' Site 3

Site 3 was a three-roadway shortwall development section, and the sonic probe monitoring holes were drilled in the centre roadway. A road header together with the shuttle cars were used to mine No 2B Seam in the Witbank Coalfield. The stress mapping technique showed that there was no apparent horizontal stress driven damage in this section. In general, the pillar and roof conditions were excellent. The installation and performance of support were also found to be excellent in the section.

The road-header experiment sequence was used at this site. After the installation of the first hole was completed, the initial reading was taken from this hole. The face was then advanced by 8.0 m in full bord width and the second hole drilled and instrumented with sonic probe anchors. Readings were taken from both holes. The further readings from both holes were taken after the completion of 16 m face advance and 96-hours stand-up time. Before the experiment was completed, two more readings were taken, after approximately 20 m (7 days after the support installation) and 100 m face advance.

The results obtained from both holes during the experiment in Site 2 are presented in Figure 4-8. The results indicated that while No 1 hole was stable during the experiment, No 2 hole showed a 3.0 mm displacement after 8.0 m face advance (16 m full length completed) and 48-hours stand-up time. No further displacement was recorded in No 2 hole. Similar to Site 2, an

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intersection was developed between the two sonic probe monitoring holes after the second last reading (seven days after support installation as indicated in Figure 4-8); however, no further displacement took place after this stress change.

The summary of the site performance in this site is given in Table 4-5.

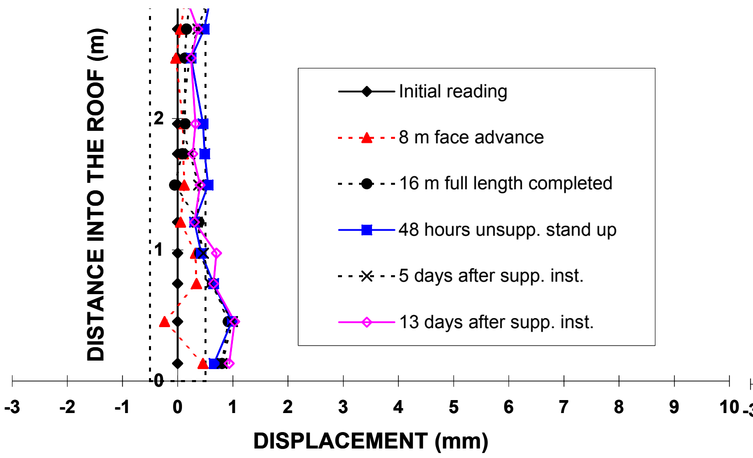
***Table 4-5 Site performance Colliery 'A' Site 3***

| | | | |
|---|---|---|---|
| Coalfield: | Witbank | Seam: | No 2 |
| Site: | Three | Positions: | Roadway |
| Road widths: | 6.1 m | Pillar widths: | 30 x 10 m |
| Mining height: | 3.0 m | Depth: | 53 m |
| Mining method: | Road header and shuttle cars | | |
| Cut-out distance | 16.7 m | | |
| Roof strata: | 0.28 m laminated sandstone overlain by 0.85 m grit which overlain by thick sandstone (>2.0 m) | | |
| Support: | 1.8 m x 16 mm Re-bar, full column resin in 22 mm hole.<br>Support density 0.44 bolt/$m^2$. | | |
| Performance: | No 1 hole was stable during the experiment.<br><br>No 2 hole showed approximately 3.0 mm dilation after 8.0 m face advance (16 m full length completed) and 48-hours stand-up time which took place at the grit/sandstone interface, 1.0 m into the roof. No further displacement was recorded in No 2 hole, even after 100 m face advance. | | |

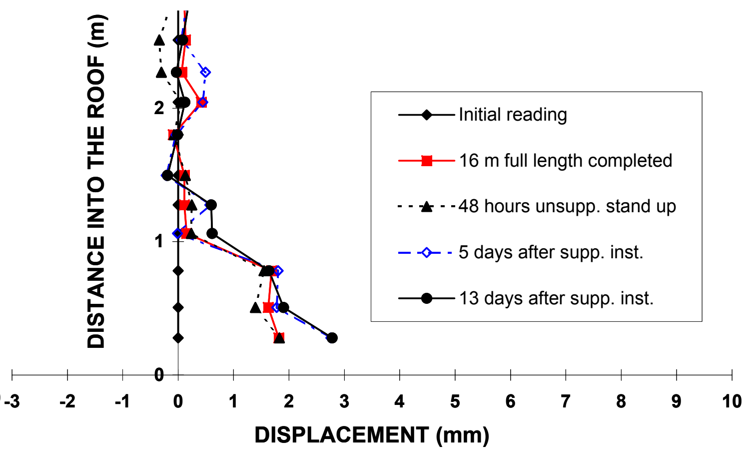
WITBANK COALFIELD
No 2B Seam

## COLLIERY "A" - SITE 1 (TEST1)
(CUT OUT DISTANCE 16 m)

### HOLE 1




### HOLE 2




### Immediate roof lithology (not scaled)

| DEPTH INTO THE ROOF (m) | SECTION | WIDTH (m) | RECORD OF STRATA |
|---|---|---|---|
| 1.6 | | >2 | SHALE, black, fine grained, fissile |
| 0.86 | | 0.74 | SANDSTONE, white, coarse to medium |
| 0.78 | | 0.08 | COAL, dull lustrous, 10 - 40% bright |
| 0.13 | | 0.65 | GRIT/COAL LAMINAE |
| 0 | | 0.13 | GRIT, white, medium grained, coaly |

### Site performance

| | |
|---|---|
| BOLT TYPE: | OZ-BAR |
| BOLT DIAMETER (mm): | 20 |
| HOLE DIAMETER (mm): | 25 |
| BOLT LENGTH (mm): | 1500 |
| HOLE LENGTH (mm): | 1500 |
| NUMBER OF BOLTS IN A ROW: | 5 |
| DISTANCE BETWEEN THE ROWS (m): | 1.5 |
| BOLT/m$^2$ | 0.57 |
| | |
| RESIN CAPSULE DIAMETER (m): | 19 |
| RESIN TYPES | SLOW & FAST |
| NUMBER OF RESIN CAPSULES: | 3 |
| | |
| BORD WIDTH (m): | 5.8 |
| PILLAR WIDTH (m): | 9 |
| DEPTH (m): | 32 |
| MINING HEIGHT (m): | 3 |
| SAFETY FACTOR: | 5.19 |

### Cutting sequence

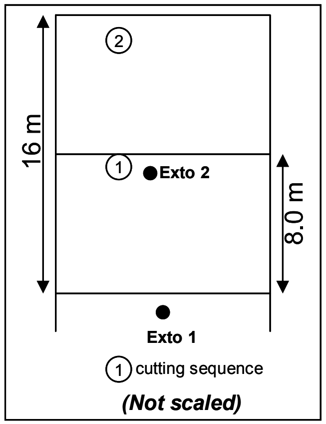
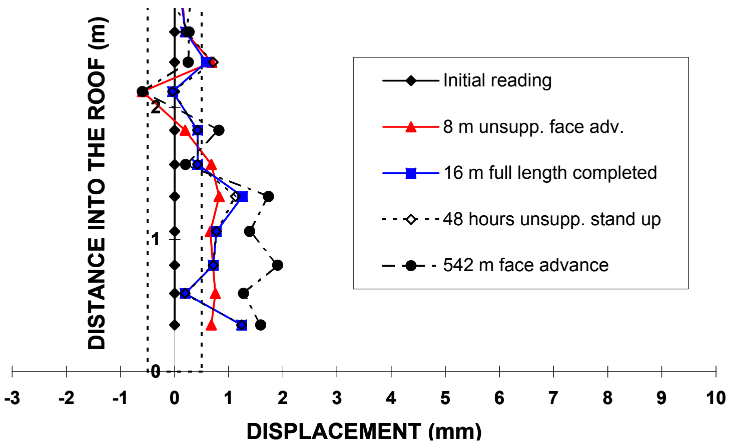



***Figure 4-5 Colliery 'A' Site 1, Test 1***

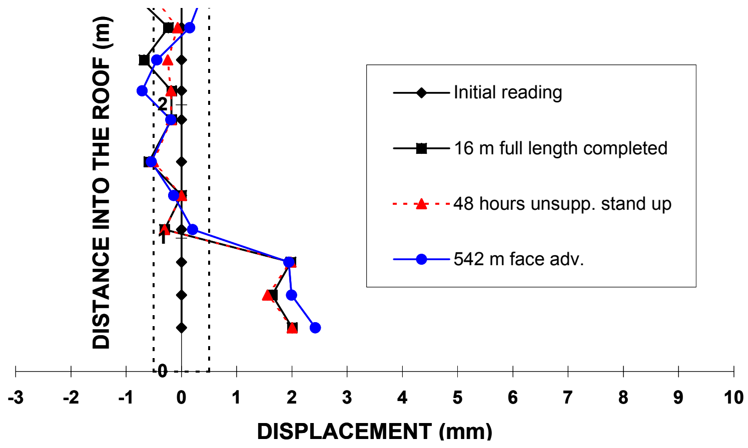
WITBANK COALFIELD
No 2B Seam

COLLIERY "A" - SITE 1 (TEST2)
(CUT OUT DISTANCE 16 m)

HOLE 1




HOLE 2




Immediate roof lithology (not scaled)

| DEPTH INTO THE ROOF (m) | SECTION | WIDTH (m) | RECORD OF STRATA |
|---|---|---|---|
| 1.6 | | >2 | SHALE, black, fine grained, fissile |
| 0.86 | | 0.74 | SANDSTONE, white, coarse to medium |
| 0.78 | | 0.08 | COAL, dull lustrous, 10 - 40% bright |
| 0.13 | | 0.65 | GRIT/COAL LAMINAE |
| 0 | | 0.13 | GRIT, white, medium grained, coaly |

Site performance

| | |
|---|---|
| BOLT TYPE: | OZ-BAR |
| BOLT DIAMETER (mm): | 20 |
| HOLE DIAMETER (mm): | 25 |
| BOLT LENGTH (mm): | 1500 |
| HOLE LENGTH (mm): | 1500 |
| NUMBER OF BOLTS IN A ROW: | 5 |
| DISTANCE BETWEEN THE ROWS (m): | 1.5 |
| BOLT/m$^2$ | 0.57 |
| | |
| RESIN CAPSULE DIAMETER (m): | 19 |
| RESIN TYPES | SLOW & FAST |
| NUMBER OF RESIN CAPSULES: | 3 |
| | |
| BORD WIDTH (m): | 5.8 |
| PILLAR WIDTH (m): | 9 |
| DEPTH (m): | 32 |
| MINING HEIGHT (m): | 3 |
| SAFETY FACTOR: | 5.19 |

Cutting sequence

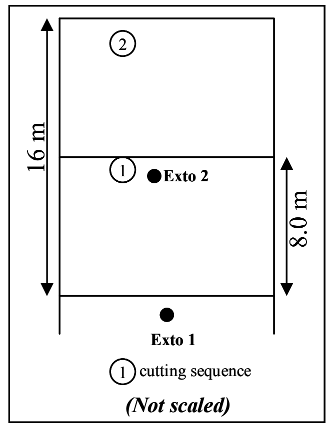
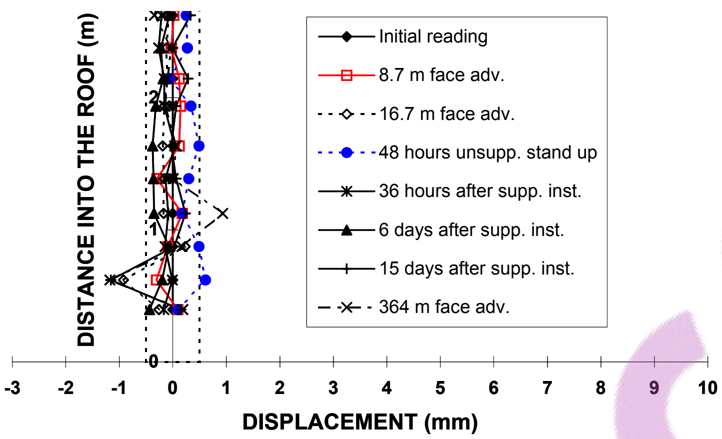



***Figure 4-6 Colliery 'A' Site 1, Test 2***

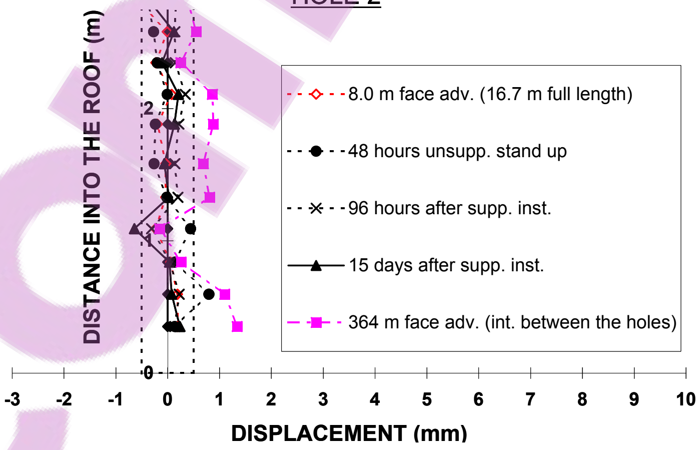
WITBANK COALFIELD
No 2B Seam

COLLIERY "A" - SITE 2
(CUT OUT DISTANCE 16.7 m)

HOLE 1




HOLE 2




Immediate roof lithology (not scaled)

| DEPTH INTO THE ROOF (m) | SECTION | WIDTH (m) | RECORD OF STRATA |
|---|---|---|---|
| 0.54 | | 1.8 | SHALE, black |
| 0 | | 0.54 | SANDSTONE, gritty, silty |

Site performance

| | |
|---|---|
| BOLT TYPE: | OZ-BAR |
| BOLT DIAMETER (mm): | 20 |
| HOLE DIAMETER (mm): | 25 |
| BOLT LENGTH (mm): | 900 |
| HOLE LENGTH (mm): | 900 |
| NUMBER OF BOLTS IN A ROW: | 4 |
| DISTANCE BETWEEN THE ROWS (m): | 1.5 |
| BOLT/$m^2$ | 0.41 |
| | |
| RESIN CAPSULE DIAMETER (m): | 19 |
| RESIN TYPES | SLOW & FAST |
| NUMBER OF RESIN CAPSULES: | 2 |
| | |
| BORD WIDTH (m): | 6.58 |
| PILLAR WIDTH (m): | 7.5 |
| DEPTH (m): | 59 |
| MINING HEIGHT (m): | 3.4 |
| SAFETY FACTOR: | 1.89 |

Cutting sequence

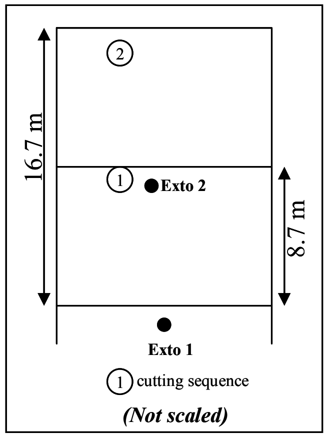
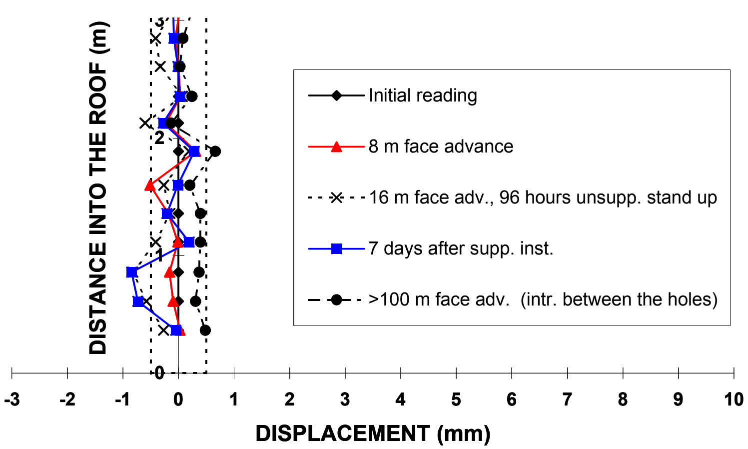



***Figure 4-7 Colliery 'A' Site 2***

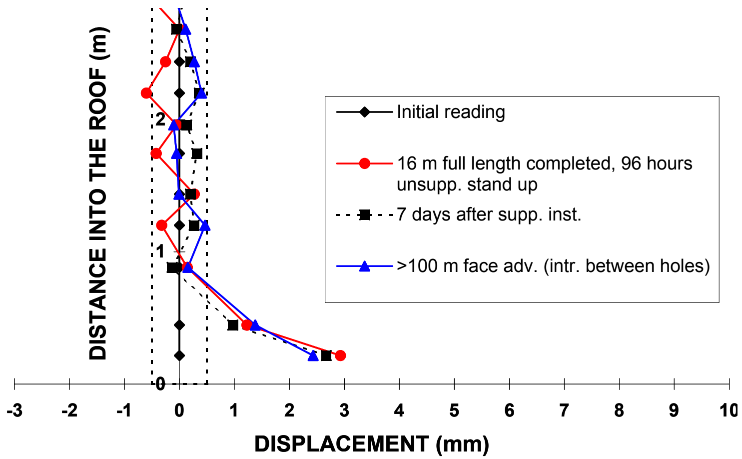
WITBANK COALFIELD
No 2B Seam

COLLIERY "A" - SITE 3
(CUT OUT DISTANCE 16 m)

HOLE 1




HOLE 2




Immediate roof lithology (not scaled)

| DEPTH INTO THE ROOF (m) | SECTION | WIDTH (m) | RECORD OF STRATA |
|---|---|---|---|
| 1.13 | | >2 | Sandstone, light grey, thinly lam. |
| 0.28 | | 0.85 | GRIT, white, very coarse grained |
| 0 | | 0.28 | SANDSTONE, dark gray, thinly laminated |

Site performance

| | |
|---|---|
| BOLT TYPE: | RE-BAR |
| BOLT DIAMETER (mm): | 16 |
| HOLE DIAMETER (mm): | 22 |
| BOLT LENGTH (mm): | 1800 |
| HOLE LENGTH (mm): | 1800 |
| NUMBER OF BOLTS IN A ROW: | 4 |
| DISTANCE BETWEEN THE ROWS (m): | 1.5 |
| BOLT/m$^2$ | 0.44 |
| | |
| RESIN CAPSULE DIAMETER (m): | 19 |
| RESIN TYPES | SLOW & FAST |
| NUMBER OF RESIN CAPSULES: | 3 |
| | |
| BORD WIDTH (m): | 6.1 |
| PILLAR WIDTH (m): | 15 |
| DEPTH (m): | 53 |
| MINING HEIGHT (m): | 3 |
| SAFETY FACTOR: | 5.09 |

Cutting sequence

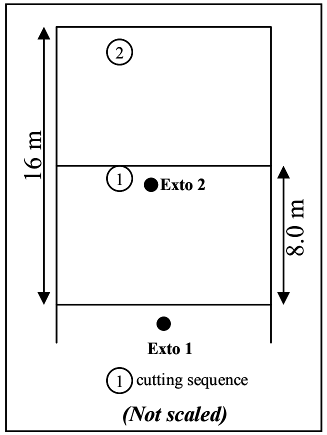



***Figure 4-8 Colliery 'A' Site 3***

## 4.5 Colliery 'B'

Three sites in three different sections were monitored at Colliery ‘B’. The colliery is situated in the Witbank Coalfield and mining is currently being conducted in the No 2 and No 5 Seams.

### 4.5.1 Colliery 'B' Site 1

Site 1 was a five-roadway, primary bord and pillar production section. The sonic probe monitoring holes were drilled in the belt road (centre roadway). A CM together with shuttle cars was used in the section to mine No 2 Seam in the Witbank Coalfield. There were no excessive stress indications in the section. While the underground dimension control was satisfactory, the roof and the pillar conditions were good.

The road header experiment sequence was used at this site. After the instrumentation of the first hole, completed at the face, the initial reading was taken. The face was then advanced by 10 m in full bord width and the second hole drilled and instrumented with sonic probe anchors. Readings were taken from both holes. The cut-out length was then completed by advancing the face by 14 m. The readings from both holes were taken again and the face was left for 48-hours unsupported in order to determine the effect of time. Further readings were taken 20 days after support installation, when the face advance was approximately 50 m.

The results showed that during the experiment both holes were stable and no displacement was recorded in either hole. The results obtained from both holes during the first experiment in Colliery 'B', Site 1 are presented in Figure 4-9. The summary of site performance in this site is given in Table 4-6.

***Table 4-6 Site performance Colliery ‘B’ Site 1***

| | | | |
|---|---|---|---|
| Coalfield: | Witbank | Seam: | No 2 |
| Site: | One | Positions: | Roadway |
| Road widths: | 6.5 m | Pillar widths: | 10.5 m |
| Mining height: | 4.4 m | Depth: | 75 m |
| Mining method: | CM and shuttle cars | | |
| Cut-out distance | 24 m | | |
| Roof strata: | 1.0 m coal overlain by 1.05 m thick sandstone | | |

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| | |
|---|---|
| Support: | 1.8 m x 16 mm Re-bar, full column resin in 22 mm hole.<br>Support density 0.15 bolt/$m^2$. |
| Performance: | Both holes showed no dilation during the experiment. This is thought to be due to the 1.0 m thick coal left in the coal. |

### 4.5.2 Colliery 'B' Site 2

While this section was a 12-roadway primary bord and pillar production section, the experiment took place in an area where the number of roadways was reduced to five. The sonic probe holes were drilled in the one-left roadway. A road header together with shuttle cars was used in the section to mine the No 2 Seam in the Witbank Coalfield. Stress mapping techniques showed that there was no excessive horizontal stress in the section. The roof and the pillar conditions were good.

The road header cutting sequence was applied in a 31 m cut-out distance in the section. After the instrumentation of the first hole was completed at the face, the initial reading was taken. The face was then advanced by 16 m at full bord width and the second hole drilled and instrumented with sonic probe anchors, and readings were taken from both holes. Advancing the face by 15 m then completed the cut-out length and readings from both holes were taken. Because of the long cut-out distance in the experiment, the face was not left unsupported for 48-hours, and the area was supported as soon as the readings were taken. The face was then advanced by a further 21 m and readings were taken from both holes.

Installation and performance of support were found to be good in the section. During the experiment both holes were stable and no movement was recorded in either hole. One reason for this can be that displacement in the roof had occurred before instrumentation of the second hole, as the length of the first face advance was 16 m. This will be investigated further in the following section.

The results obtained from both holes during the first experiment in Site 1 are presented in Figure 4-10. The summary of site performance in this site is given in Table 4-7.

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***Table 4-7 Site performance Colliery 'B' Site 2***

| | | | |
|---|---|---|---|
| Coalfield: | Witbank | Seam: | No 2 |
| Site: | Two | Positions: | Roadway |
| Road widths: | 6.7 m | Pillar widths: | 15.7 m |
| Mining height: | 4.2 m | Depth: | 44 m |

| | |
|---|---|
| Mining method: | Road header and shuttle cars |
| Cut-out distance | 31 m |
| Roof strata: | 0.856 m coal overlain by 0.65 m thick shale/siltstone |
| Support: | 1.8 m x 16 mm Re-bar, full column resin in 22 mm hole. Support density 0.15 bolt/$m^2$. |
| Performance: | Both holes showed no dilation during the experiment. |

### 4.5.3 Colliery 'B' Site 3

The section was a 17-roadway, primary bord and pillar production section, and the monitoring holes were drilled in the belt road (centre roadway in the section). A CM with shuttle cars was used in the section to mine No 5 Seam in the Witbank Coalfield. Localized bord and intersection failures and pillar-roof contact scaling in various parts of the section raised possibility of horizontal stress damage. The detailed stress mapping technique also showed that the horizontal stress was higher than the strength of the immediate rock layer. However, while excessive horizontal stress caused damage in the roof in some areas, there was no movement in the roof in the experiment site, again indicating the variable nature of the conditions.

The road header experiment sequence was used in the section. After the installation of the first hole was completed, the initial reading was taken. Then the face was advanced by 6.0 m at full width, the second hole drilled and instrumented and initial readings were taken from the second hole. The face was advanced a further 6.0 m and readings were taken from both holes. Without leaving the face for 48-hours unsupported, the installation of the support was started and readings were taken after the installation of each row of support, in order to determine the effect of bolting in the roof. The readings were taken up to a point where the support passed the second hole in the experiment site. However, because no movement took place during the 12 m cut-out distance, the effect of roof bolting could not be monitored.

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In general, the roof and the pillar conditions were good as well as the quality of support installation.

The results obtained from both holes during the first experiment in Site 1 are presented in Figure 4-11. The summary of site performance in this site is given in Table 4-8.

***Table 4-8    Site performance Colliery 'B' Site 3***

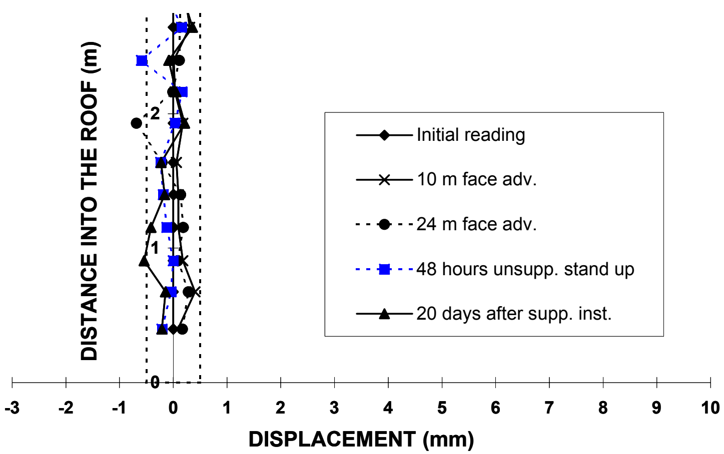
| | | | |
|---|---|---|---|
| Coalfield: | Witbank | Seam: | No 5 |
| Site: | Three | Positions: | Roadway |
| Road widths: | 6.2 m | Pillar widths: | 6.4 m |
| Mining height: | 1.9 m | Depth: | 43.7 m |

| | |
|---|---|
| Mining method: | CM and shuttle cars |
| Cut-out distance | 12 m |
| Roof strata: | 0.27 m thick interlaminated sandstone overlain by 1.27 m massive sandstone. |
| Support: | 0.9 m x 20 mm OZ-Bar, full column resin in 25 mm hole.<br>Support density 0.24 bolt/$m^2$. |
| Performance: | Both holes showed no dilation during the experiment. |

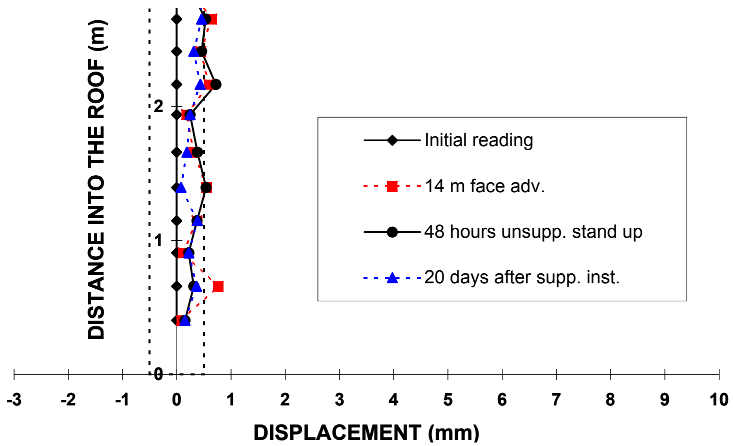
WITBANK COALFIELD
No 2 Seam

COLLIERY "B" - SITE 1
(CUT OUT DISTANCE 24 m)

HOLE 1




HOLE 2




Immediate roof lithology (not scaled)

| DEPTH INTO THE ROOF (m) | SECTION | WIDTH (m) | RECORD OF STRATA |
|---|---|---|---|
| 1.46 | | 0.59 | SANDSTONE, grey, fine grained, micaceous bedding planes |
| 1 | | 0.46 | SANDSTONE, shaly, with bioturbation |
| 0 | | 1 | COAL |

Site performance

| | |
|---|---|
| BOLT TYPE: | RE-BAR |
| BOLT DIAMETER (mm): | 16 |
| HOLE DIAMETER (mm): | 24 |
| BOLT LENGTH (mm): | 1800 |
| HOLE LENGTH (mm): | 1800 |
| NUMBER OF BOLTS IN A ROW: | 2 |
| DISTANCE BETWEEN THE ROWS (m): | 2 |
| BOLT/m$^2$ | 0.15 |
| | |
| RESIN CAPSULE DIAMETER (m): | 19 |
| RESIN TYPES | SLOW & FAST |
| NUMBER OF RESIN CAPSULES: | 3 |
| | |
| BORD WIDTH (m): | 6.5 |
| PILLAR WIDTH (m): | 10.5 |
| DEPTH (m): | 75 |
| MINING HEIGHT (m): | 4.4 |
| SAFETY FACTOR: | 1.86 |

Cutting sequence

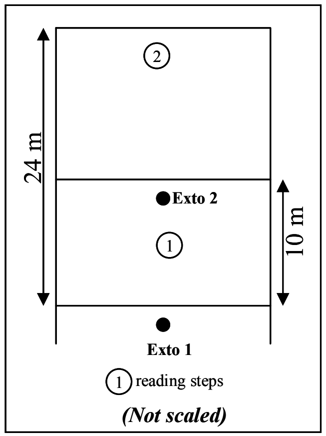
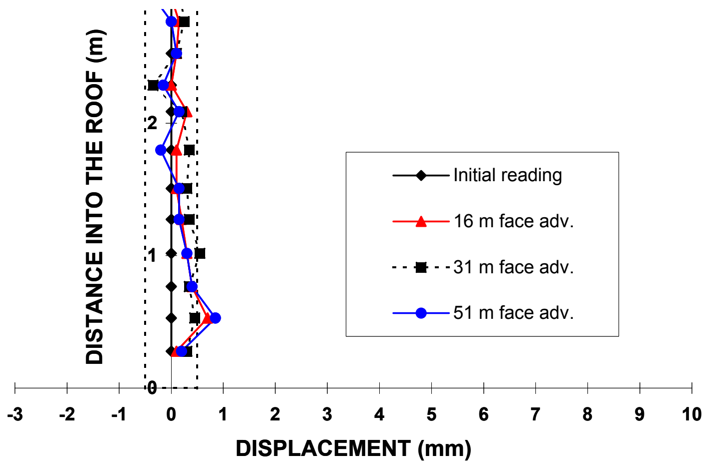



***Figure 4-9 Colliery 'B' Site 1***

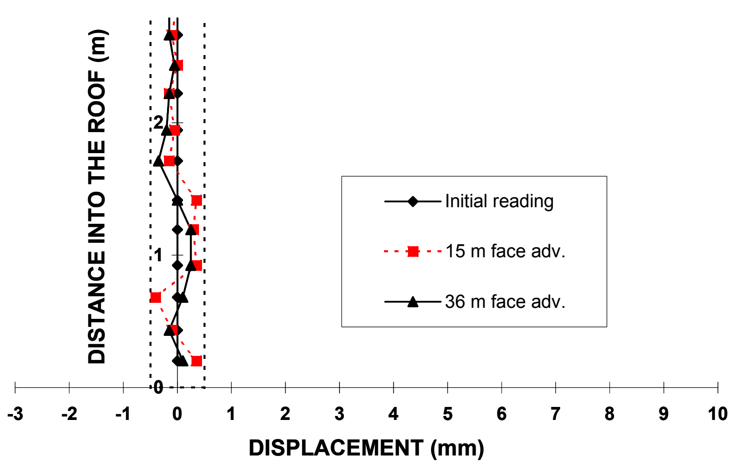
WITBANK COALFIELD
No 2 Seam

COLLIERY "B" - SITE 2
(CUT OUT DISTANCE 31 m)

HOLE 1




HOLE 2




Immediate roof lithology (not scaled)

| DEPTH INTO THE ROOF (m) | SECTION | WIDTH (m) | RECORD OF STRATA |
|---|---|---|---|
| 1 | | 0.65 | SHALE/SILTSTONE, dark greyish-black car. Mic. |
| 0 | | 0.85 | COAL |

Site performance

| | |
|---|---|
| BOLT TYPE: | RE-BAR |
| BOLT DIAMETER (mm): | 16 |
| HOLE DIAMETER (mm): | 24 |
| BOLT LENGTH (mm): | 1800 |
| HOLE LENGTH (mm): | 1800 |
| NUMBER OF BOLTS IN A ROW: | 2 |
| DISTANCE BETWEEN THE ROWS (m): | 2 |
| BOLT/$m^2$ | 0.15 |
| | |
| RESIN CAPSULE DIAMETER (m): | 19 |
| RESIN TYPES | SLOW & FAST |
| NUMBER OF RESIN CAPSULES: | 3 |
| | |
| BORD WIDTH (m): | 6.7 |
| PILLAR WIDTH (m): | 15.7 |
| DEPTH (m): | 44 |
| MINING HEIGHT (m): | 4.2 |
| SAFETY FACTOR: | 4.85 |

Cutting sequence

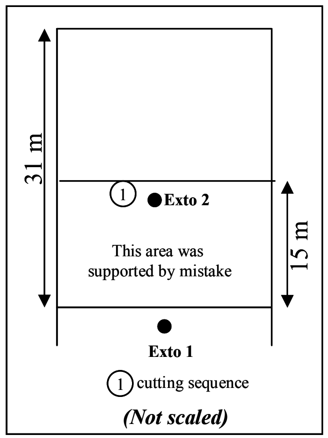
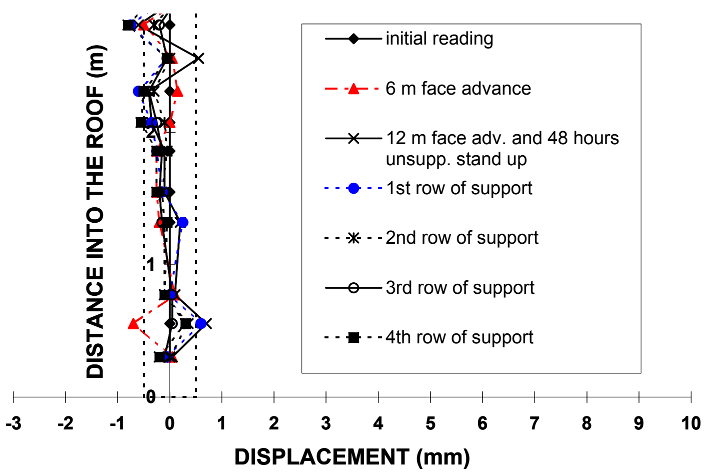



***Figure 4-10 Colliery 'B' Site 2***

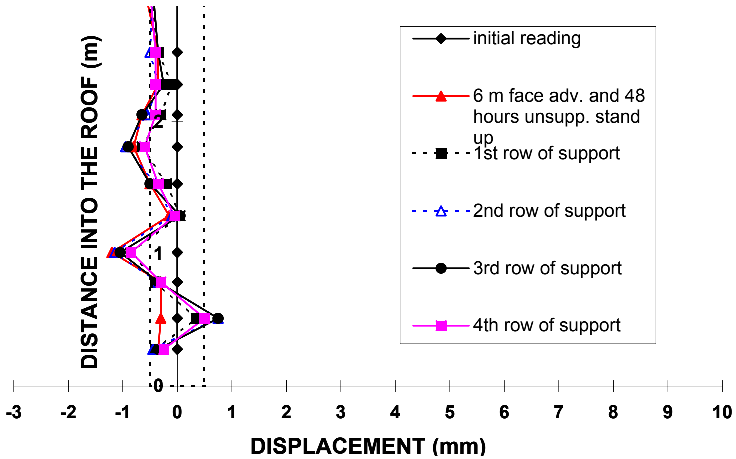
WITBANK COALFIELD
No 5 Seam

COLLIERY "B" - SITE 3
(CUT OUT DISTANCE 12 m)

HOLE 1




HOLE 2




Immediate roof lithology (not scaled)

| DEPTH INTO THE ROOF (m) | SECTION | WIDTH (m) | RECORD OF STRATA |
|---|---|---|---|
| 1.54 | | > 9 | SHALE / SILTDTONE |
| 0.27 | | 1.27 | SANDSTONE, greenish-grey, coarse grained, massive |
| 0 | | 0.27 | SANDSTONE, light grey, medium grained, interlaminated |

Site performance

| | |
|---|---|
| BOLT TYPE: | OZ-BAR |
| BOLT DIAMETER (mm): | 20 |
| HOLE DIAMETER (mm): | 26 |
| BOLT LENGTH (mm): | 900 |
| HOLE LENGTH (mm): | 900 |
| NUMBER OF BOLTS IN A ROW: | 3 |
| DISTANCE BETWEEN THE ROWS (m): | 2 |
| BOLT/$m^2$ | 0.24 |
| | |
| RESIN CAPSULE DIAMETER (m): | 19 |
| RESIN TYPES | SLOW & FAST |
| NUMBER OF RESIN CAPSULES: | 2 |
| | |
| BORD WIDTH (m): | 6.2 |
| PILLAR WIDTH (m): | 6.4 |
| DEPTH (m): | 43.74 |
| MINING HEIGHT (m): | 1.66 |
| SAFETY FACTOR: | 3.56 |

Cutting sequence

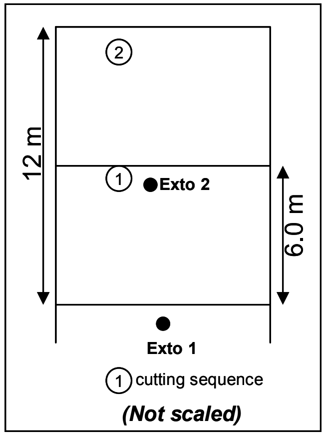



***Figure 4-11 Colliery 'B' Site 3***

## 4.6 Colliery 'C'

Three sites in three different sections were monitored at Colliery ‘C’. The colliery is situated in the Highveld Coalfield and mining is currently underway in the No 4 Seam.

### 4.6.1 Colliery 'C' Site 1

Site 1 was a 17-roadway, primary bord and pillar production section. The sonic probe monitoring holes were drilled in the two-right roadway. A road header together with shuttle cars was used in the section. There were no excessive stress indications in the section. The roof and pillar conditions were good as well as the quality of support installation.

The road header experiment sequence was used at this site. After the instrumentation of the first hole was completed at the face, the initial reading was taken. The face was then advanced by 6.0 m in full bord width and the second hole drilled and instrumented with sonic probe anchors. Readings were taken from both holes. Advancing the face by 6.0 m then completed the cut-out length, and the face was left for 48-hours unsupported. The readings from both holes were taken. A further reading was taken four days after the support installation, when the face was not advanced. The experiment was completed by taking one last reading when the face advance was 64 m.

The results showed that while hole No 1 was stable throughout the experiment, No 2 hole showed 1.5 mm dilation, which took place approximately 1.0 m into the roof at the coal/mudstone laminae and sandstone contact. The results obtained from both holes during the first experiment in Colliery 'C', Site 1, are presented in Figure 4-12. The summary of site performance in this site is given in Table 4-9.

***Table 4-9 Site performance Colliery 'C' Site 1***

| | | | |
|---|---|---|---|
| Coalfield: | Highveld | Seam: | No 4 |
| Site: | One | Positions: | Roadway |
| Road widths: | 7.0 m | Pillar widths: | 10 m |
| Mining height: | 4.3 m | Depth: | 54.2 m |

| | |
|---|---|
| Mining method: | Road header and shuttle cars |
| Cut-out distance | 12 m |

| | |
|---|---|
| Roof strata: | 0.94 m thick coal/mudstone laminae overlain by 2.25 m thick |

shale/sandstone

Support: 1.8 m x 16 mm Re-bar, partial column resin in 22 mm hole. Support density 0.14 bolt/$m^2$.

Performance: While No 1 hole was stable throughout the experiment. No 2 hole showed 1.5 mm dilation, at 1.0 m into the roof at the coal/mudstone laminae and sandstone contact. This movement took place after 6 m face advance (12 m full length completed) and 48-hours stand-up time. No further dilation was recorded even after 64 m face advance.

#### 4.6.2 Colliery 'C' Site 2

Site 1 was an 11-roadway, primary bord and pillar production section. The sonic probe monitoring holes were drilled in the centre roadway. A road header together with shuttle cars was used in the section. There were no excessive stress indications in the section. A major problem observed in support installation was the overdrilling of boltholes.

The road header experiment sequence was used at this site. After the instrumentation of the first hole was completed, the initial reading was taken. The face was then advanced by 6.0 m in full bord width and the second hole drilled and instrumented with sonic probe anchors. Readings were taken from both holes. Following readings were taken once the face was advanced by 6.0 m and after a 48-hour unsupported stand-up time period. Further readings were then taken after the area was supported at 60 m and the face was advanced by 200 m.

The results obtained from both holes during the first experiment in Colliery 'C', Site 2, are presented in Figure 4-13. The summary of site performance in this site is given in Table 4-10. Figure 4-13 shows that the total dilation in No 1 hole was 1.0 mm at the skin anchor. Initial 0.9 mm dilation, 0.5 m into the roof, took place after the face was advanced by 6.0 m. A further 0.1 mm movement, 1.0 m into the roof, was recorded after the completion of 12 m face advance and 48-hours stand-up time. After the support installation was completed, the results from No 1 hole indicated that there had been an upwards movement into the roof. Initially, this behaviour was thought to be due to roof bolting, which took place after the unsupported stand-up time. However, the roof bolting should affect the roof skin first before influencing movement further into the roof. As can be seen from the figure, the roof skin deflected 1.0 mm during the experiment. Therefore, it was decided that this movement was an anomaly and the reading may be discarded.

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The total dilation in No 2 hole was 1.0 mm, at 1.0 m into the roof, at the coal/mudstone and shale/sandstone interface. This movement took place after 6.0 m face advance (12 m full length completed) and 48-hour stand-up time. No further dilation was recorded in hole No 2.

***Table 4-10 Site performance Colliery 'C' Site 2***

| | | | |
|---|---|---|---|
| Coalfield: | Highveld | Seam: | No 4 |
| Site: | Two | Positions: | Roadway |
| Road widths: | 6.7 m | Pillar widths: | 12 m |
| Mining height: | 4.7 m | Depth: | 70 m |

| | |
|---|---|
| Mining method: | Road header and shuttle cars |
| Cut-out distance | 12 m |
| Roof strata: | 0.8 m mudstone/coal/sandstone laminae overlain by 2.3 m thick shale/sandstone |
| Support: | 1.8 m x 16 mm Re-bar, partial column resin in 22 mm hole.<br>Support density 0.15 bolt/m$^2$. |
| Performance: | Both holes showed 1.0 mm dilation after the 12 m cut-out length and 48-hours unsupported stand-up time completed. This movement took place 0.5 m and 1.0 m into the roof respectively in No 1 and No 2 holes. Both holes stabilised and no further dilation was recorded even after 200 m face advance. |

### 4.6.3 Colliery 'C' Site 3

Site 1 was an 11-roadway, primary bord and pillar production section. The sonic probe monitoring holes were drilled in the centre roadway. A road header together with shuttle cars was used in the section to mine the No 4 Seam in the Highveld Coalfield. There were no excessive stress indicators in the section. Under drilling of bolt holes was observed, however, in general support installation and performance were good.

The road header experiment sequence was used in the experiment to monitor the 18 m cut-out distance. After the installation of the first hole was completed at the face, the initial reading was taken. The face was then advanced by 6.0 m in full bord width and the second hole drilled and instrumented with sonic probe anchors. Readings were taken from both holes. Further readings were taken once the face was advanced by 12 m and after the 48-hours unsupported stand-up

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time period. Further readings were then taken after the area was supported and while the face was at 34 m and at 143 m.

The results obtained from both holes during the first experiment in Colliery 'C', Site 3 are presented in Figure 4-14. The summary of site performance in this site is given in Table 4-11. Figure 4-14 indicates that both holes, No 1 and No 2 hole, were stable during the experiment. Although approximately 0.5 mm dilation was recorded in No 2 hole, it was within the accuracy of the system.

***Table 4-11 Site performance Colliery 'C' Site 3***

| | | | |
|---|---|---|---|
| Coalfield: | Highveld | Seam: | No 4 |
| Site: | Three | Positions: | Roadway |
| Road widths: | 6.1 m | Pillar widths: | 9.0 m |
| Mining height: | 4.3 m | Depth: | 61 m |

| | |
|---|---|
| Mining method: | Road header and shuttle cars |
| Cut-out distance | 18 m |
| Roof strata: | 0.83 m shale/coal/sandstone laminae overlain by 2 m thick shale/sandstone |
| Support: | 1.8 m x 16 mm Re-bar, partial column resin in 24 mm hole.<br>Support density 0.16 bolt/m$^2$. |
| Performance: | While No 1 hole showed no dilation throughout the experiment, No 2 hole showed 0.5 mm dilation, approximately 1.1 m into the roof, which was the within the accuracy of the system. |

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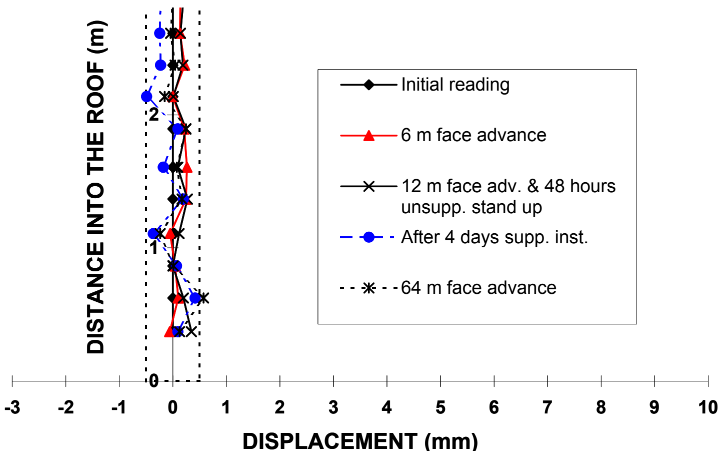
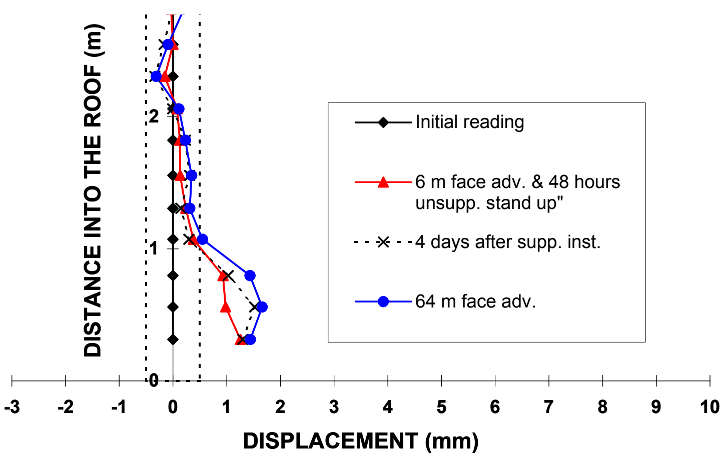
HIGHVELD COALFIELD
No 4 Seam

COLLIERY "C" - SITE 1
(CUT OUT DISTANCE 12 m)

HOLE 1




HOLE 2




Immediate roof lithology (not scaled)

| DEPTH INTO THE ROOF (m) | SECTION | WIDTH (m) | RECORD OF STRATA |
|---|---|---|---|
| 0.94 | | >2.25 | SHALE/SANDSTONE, light grey, interbedded |
| 0.71 | | 0.23 | SHALE, dark grey, silty |
| 0.56 | | 0.15 | COAL |
| 0.39 | | 0.17 | MUDSTONE |
| 0.24 | | 0.15 | COAL |
| 0 | | 0.24 | MUDSTONE |

Site performance

| | |
|---|---|
| BOLT TYPE: | RE-BAR |
| BOLT DIAMETER (mm): | 16 |
| HOLE DIAMETER (mm): | 24 |
| BOLT LENGTH (mm): | 1800 |
| HOLE LENGTH (mm): | 1750 |
| NUMBER OF BOLTS IN A ROW: | 2 |
| DISTANCE BETWEEN THE ROWS (m): | 2 |
| BOLT/m$^2$ | 0.14 |
| | |
| RESIN CAPSULE DIAMETER (m): | 19 |
| RESIN TYPES | SLOW & FAST |
| NUMBER OF RESIN CAPSULES: | 2 |
| | |
| BORD WIDTH (m): | 7.03 |
| PILLAR WIDTH (m): | 10 |
| DEPTH (m): | 54.19 |
| MINING HEIGHT (m): | 4.3 |
| SAFETY FACTOR: | 2.33 |

Cutting sequence

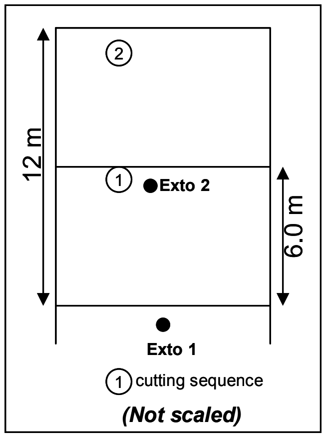
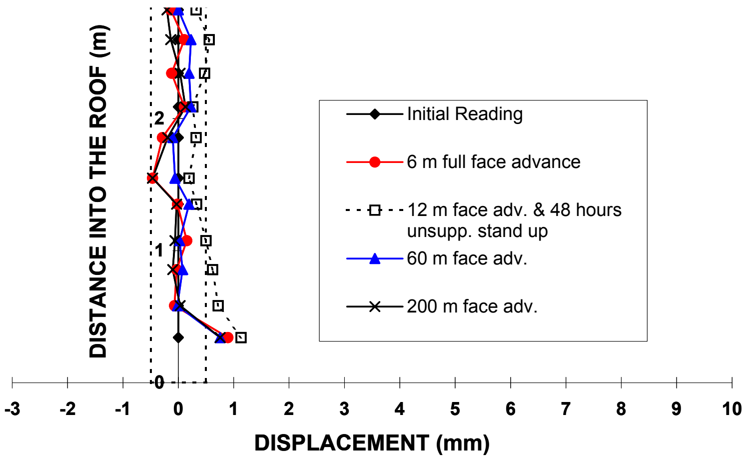
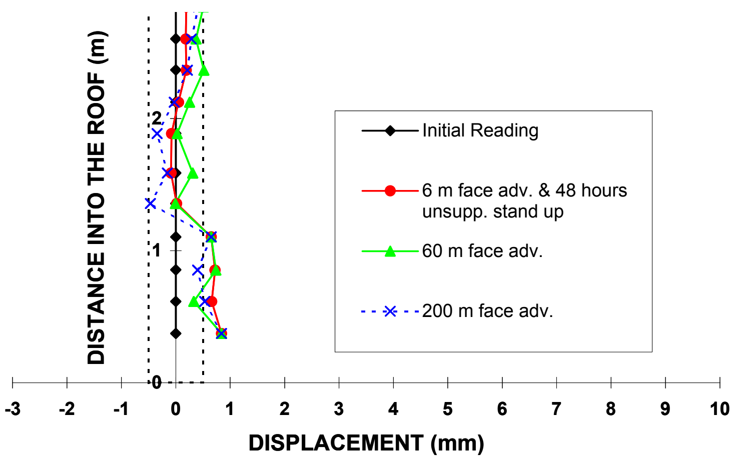



***Figure 4-12 Colliery 'C' Site 1***

HIGHVELD COALFIELD
No 4 Seam

COLLIERY "C" - SITE 2
(CUT OUT DISTANCE 12 m)




Immediate roof lithology (not scaled)

| DEPTH INTO THE ROOF (m) | SECTION | WIDTH (m) | RECORD OF STRATA |
|---|---|---|---|
| 0.83 | | 2.3 | SHALE/SANDSTONE, laminated, grey |
| 0.79 | | 0.04 | COAL |
| 0.59 | | 0.2 | SANDSTONE, light green fine grained |
| 0.32 | | 0.27 | COAL |
| 0 | | 0.32 | MUDSTONE |

Site performance

| | |
|---|---|
| BOLT TYPE: | RE-BAR |
| BOLT DIAMETER (mm): | 16 |
| HOLE DIAMETER (mm): | 21 |
| BOLT LENGTH (mm): | 1800 |
| HOLE LENGTH (mm): | 1800 |
| NUMBER OF BOLTS IN A ROW: | 2 |
| DISTANCE BETWEEN THE ROWS (m): | 2 |
| BOLT/m$^2$ | 0.15 |
| | |
| RESIN CAPSULE DIAMETER (m): | 19 |
| RESIN TYPES | FAST |
| NUMBER OF RESIN CAPSULES: | 1 |
| | |
| BORD WIDTH (m): | 6.7 |
| PILLAR WIDTH (m): | 12 |
| DEPTH (m): | 70 |
| MINING HEIGHT (m): | 4.7 |
| SAFETY FACTOR: | 2.16 |

Cutting sequence

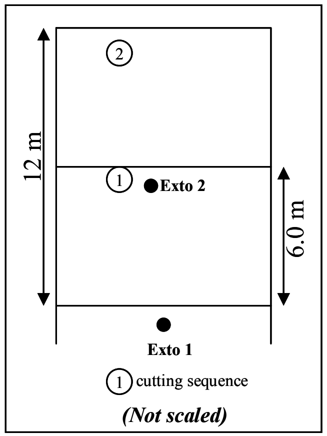
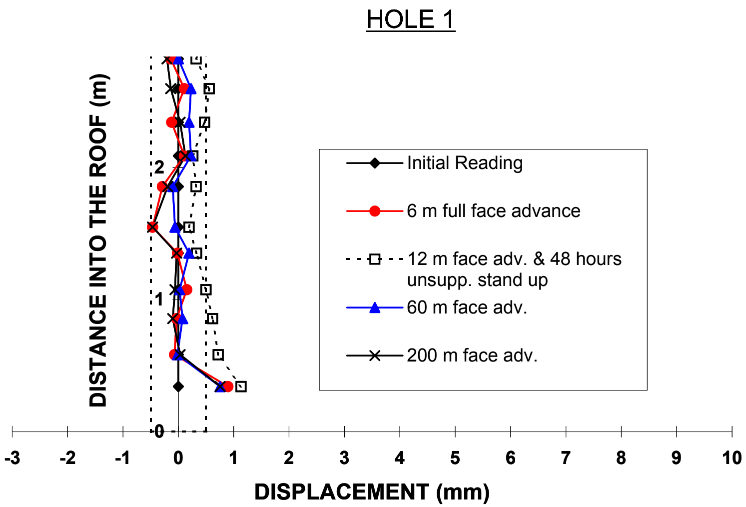
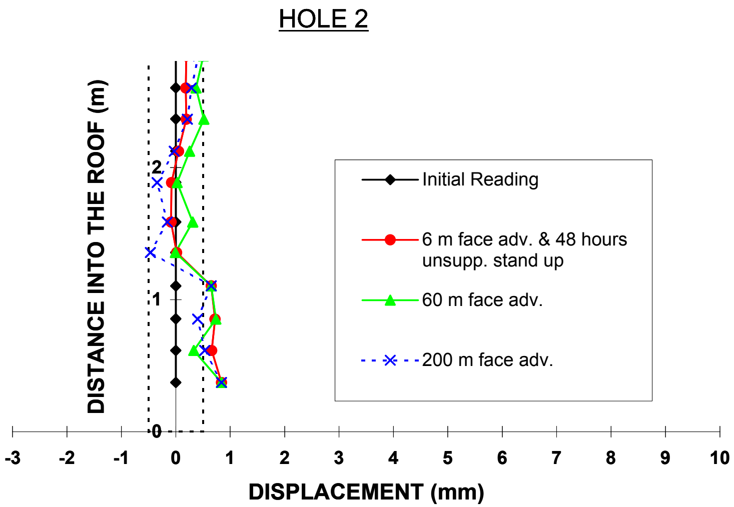



***Figure 4-13 Colliery 'C' Site 2***

HIGHVELD COALFIELD
No 4 Seam

COLLIERY "C" - SITE 2
(CUT OUT DISTANCE 12 m)




Immediate roof lithology (not scaled)

| DEPTH INTO THE ROOF (m) | SECTION | WIDTH (m) | RECORD OF STRATA |
|---|---|---|---|
| 0.83 | | 2.3 | SHALE/SANDSTONE, laminated, grey |
| 0.79 | | 0.04 | COAL |
| 0.59 | | 0.2 | SANDSTONE, light green fine grained |
| 0.32 | | 0.27 | COAL |
| 0 | | 0.32 | MUDSTONE |

Site performance

| | |
|---|---|
| BOLT TYPE: | RE-BAR |
| BOLT DIAMETER (mm): | 16 |
| HOLE DIAMETER (mm): | 21 |
| BOLT LENGTH (mm): | 1800 |
| HOLE LENGTH (mm): | 1800 |
| NUMBER OF BOLTS IN A ROW: | 2 |
| DISTANCE BETWEEN THE ROWS (m): | 2 |
| BOLT/m$^2$ | 0.15 |
| | |
| RESIN CAPSULE DIAMETER (m): | 19 |
| RESIN TYPES | FAST |
| NUMBER OF RESIN CAPSULES: | 1 |
| | |
| BORD WIDTH (m): | 6.7 |
| PILLAR WIDTH (m): | 12 |
| DEPTH (m): | 70 |
| MINING HEIGHT (m): | 4.7 |
| SAFETY FACTOR: | 2.16 |

Cutting sequence

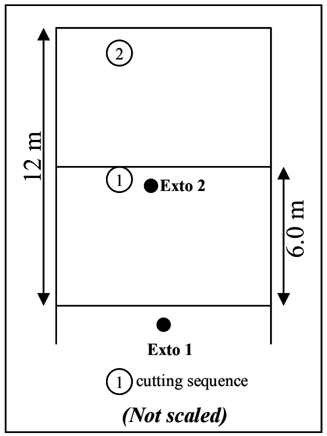



***Figure 4-14 Colliery 'C' Site 3***

## 4.7 Colliery 'D'

One site was monitored at Colliery ‘D’. The colliery is situated in the Witbank Coalfield and mining the No 2A Seam.

### 4.7.1 Colliery 'D' Site 1

Site 1 was a seven-roadway, primary bord and pillar production section. The sonic probe monitoring holes were drilled in the centre roadway. A remote controlled road header together with shuttle cars was used in the section. While there were no excessive stress indicators in the section, many geological discontinuities were present in the roof and pillars. However, the pillar, roof and underground dimension control were good.

The road header experiment sequence was used in the experiment. In order to monitor the roof displacement profile, three sonic probe-monitoring holes were used. These holes were situated at the face, 6.0 m and 8.0 m into the advancing section. After the installation of the first hole was completed, the initial reading was taken. The face was then advanced by 6.0 m in full bord width and the second hole drilled and instrumented with sonic probe anchors. Readings were taken from both holes. The third hole was then drilled after the face was advanced by 2.0 m, and readings were taken from all three holes. The final cut-out distance was reached after the face was advanced by a further 8.0 m. Because of the amount of time potentially spent under an unsupported roof during the drilling and instrumentation of holes, it was decided to support the roof immediately after the 16 m cut-out distance was completed. An attempt was made to monitor the effect of roof bolting. Therefore, readings were taken after installation of each row of support up to a point where the last row of support passed the third monitoring hole. The last readings were taken from all three holes when the face was advanced by 60 m. The results obtained from both holes during the first experiment in Colliery 'D', Site 1 are presented in Figure 4-15. The summary of site performance in this site is given in Table 4-12.

Figure 4-15 shows that while hole No 1 showed the least dilation of 2.0 mm, 2.5 mm dilation was recorded in both No 2 and No 3 holes. In all three holes the dilations took place approximately 1.5 m into the roof, and before the support was installed. Also, holes No 1 and No 3 showed a further 0.5 mm dilation after the face advanced by 60 m. It was noted that this deformation could be due to stress changes caused by development of an intersection 0.5 m away from the third hole.

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As mentioned earlier, one of the aims of this experiment was to monitor the effect of roof bolting and/or tensioning in the roof. While it was not possible to observe the effect of roof bolting in the No 1 or No 3 holes, because the initial displacements took place at the bolted horizon as a whole beam, Hole No 2 presented an ideal site to attempt to determine the effect of the installation of the roof bolts. The installation of the bolts appeared to have little if any effect on the bed separation already evident within the bolted interval as can be seen in Figure 4-15. However, as this information came from only one monitoring hole, it cannot be concluded that this is typical and that the installation of pre-tensioned roof bolts has no remedial effects on pre-existing openings within the bolt horizon.

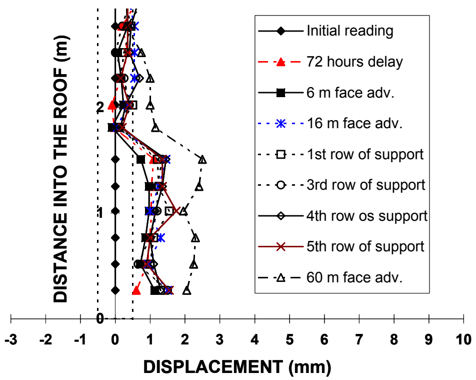
***Table 4-12 Site performance Colliery 'D' Site 1***

| | | | |
|---|---|---|---|
| Coalfield: | Witbank | Seam: | No 2A |
| Site: | One | Positions: | Roadway |
| Road widths: | 6.6 m | Pillar widths: | 8.2 m |
| Mining height: | 4.2 m | Depth: | 53 m |
| Mining method: | Road header and shuttle cars | | |
| Cut-out distance | 16 m | | |
| Roof strata: | 1.83 thick laminated coal/shale | | |
| Support: | 1.5 m x 16 mm Re-bar, partial column resin in 22 mm hole. Support density 0.23 bolt/$m^2$. | | |
| Performance: | While No 1 and No 3 holes showed 2.5 mm, No 2 hole showed 2.0 mm displacement. | | |

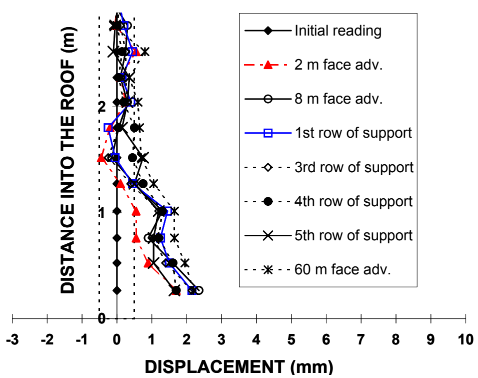
WITBANK COALFIELD
No 2A Seam

## COLLIERY "D" – SITE 1
(CUT OUT DISTANCE 16 m)

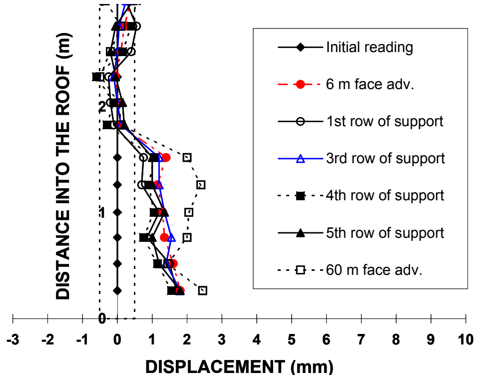
### HOLE 1




### HOLE 2




### HOLE 3




### Immediate roof lithology (not scaled)

| DEPTH INTO THE ROOF (m) | SECTION | WIDTH (m) | RECORD OF STRATA |
|---|---|---|---|
| 1.83 | | 0.65 | SHALE, carbonaceous |
| 0 | | 1.83 | COAL, SHALY COAL, bright bands |

### Site performance

| | |
|---|---|
| BOLT TYPE: | Resin Point Anchor |
| BOLT DIAMETER (mm): | 16 |
| HOLE DIAMETER (mm): | 22 |
| BOLT LENGTH (mm): | 1500 |
| HOLE LENGTH (mm): | 1500 |
| NUMBER OF BOLTS IN A ROW: | 3 |
| DISTANCE BETWEEN THE ROWS (m): | 2 |
| BOLT/m$^2$ | 0.23 |
| | |
| RESIN CAPSULE DIAMETER (m): | 19 |
| RESIN TYPES | FAST |
| NUMBER OF RESIN CAPSULES: | 2 |
| | |
| BORD WIDTH (m): | 6.6 |
| PILLAR WIDTH (m): | 8.2 |
| DEPTH (m): | 53 |
| MINING HEIGHT (m): | 4.2 |
| SAFETY FACTOR: | 2.03 |

### Cutting sequence

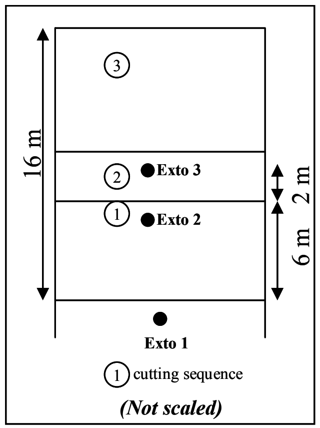



***Figure 4-15 Colliery 'D' Site 1***

## 4.8 Colliery 'E'

One site was monitored at Colliery ‘E’. The colliery is situated in the Highveld Coalfield and mining is being carried out on No 4 Lower Seam.

### 4.8.1 Colliery 'E' Site 1

Site 1 was a seven-roadway, primary bord and pillar production section. The sonic probe monitoring holes were drilled in the centre-roadway. A remote controlled CM together with shuttle cars was used in the section. There were no excessive horizontal stress indicators in the section. The installation and performance of support was excellent.

The road header experiment sequence was used at this site. After the installation of the first hole was completed at the face, the initial reading was taken. The face was then advanced by 12 m in full bord width and the second hole drilled and instrumented with sonic probe anchors. Readings were taken from both holes. The final cut-out distance was reached after the face was advanced by a further 12 m, and the face was left unsupported for 40 hours. Further readings were taken from both holes. The last reading was taken after the face was advanced by a further 13 m. The results obtained from both holes during the experiment in Colliery 'E' are presented in Figure 4-16. The summary of site performance in this site is given in Table 4-13.

No 1 and No 2 holes showed dilation of 1.0 mm and 5.0 mm respectively, in both cases extending 0.3 m into the roof. In both monitoring holes, the displacements took place at the coal/sandstone/siltstone contact after the completion of 24 m cut-out length and 48-hours stand-up time.

***Table 4-13 Site performance Colliery 'E' Site 1***

| | | | |
|---|---|---|---|
| Coalfield: | Highveld | Seam: | 4 Lower |
| Site: | One | Positions: | Roadway |
| Road widths: | 7.0 m | Pillar widths: | 25 m |
| Mining height: | 3.8 m | Depth: | 178 m |
| Mining method: | CM and shuttle cars | | |
| Cut-out distance | 24 m | | |
| Roof strata: | 0.35 m thick coal/sandstone laminae overlain by 0.65 m thick siltstone, and thick gritstone | | |

Support: 1.5 m x 20 mm Re-bar, partial column resin in 25 mm hole. Support density 0.24 bolt/$m^2$.

Performance: A maximum of 1.0 mm was recorded in No 1 hole.

No 2 showed 5.0 mm displacement, which was the largest of all the monitoring sites. Displacements in both holes took place 0.3 m into the roof at the coal/sandstone and siltstone interface.

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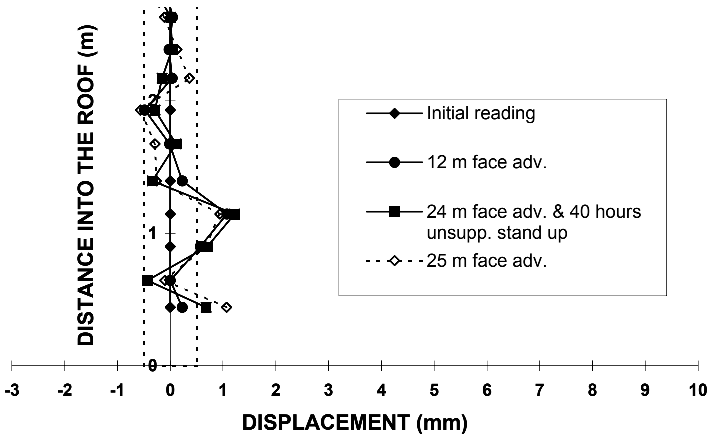
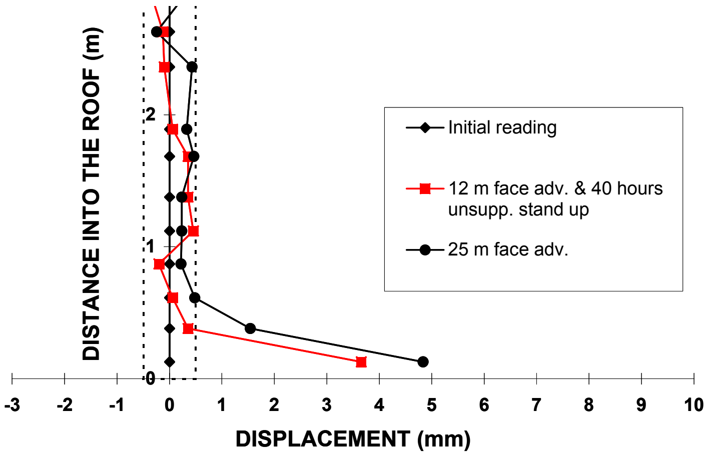
HIGHVELD COALFIELD
No 4 Lower

COLLIERY "E" - SITE 1
(CUT OUT DISTANCE 24 m)

HOLE 1




HOLE 2




Immediate roof lithology (not scaled)

| DEPTH INTO THE ROOF (m) | SECTION | WIDTH (m) | RECORD OF STRATA |
|---|---|---|---|
| 1 | | >3.9 | GRITSTONE |
| 0.35 | | 0.65 | SILTSTONE |
| 0.27 | | 0.08 | COAL |
| 0.15 | | 0.12 | SANDSTONE |
| 0 | | 0.15 | COAL, shale |

Site performance

| | |
|---|---|
| BOLT TYPE: | RE-BAR |
| BOLT DIAMETER (mm): | 20 |
| HOLE DIAMETER (mm): | 24 |
| BOLT LENGTH (mm): | 1500 |
| HOLE LENGTH (mm): | 1500 |
| NUMBER OF BOLTS IN A ROW: | 5 |
| DISTANCE BETWEEN THE ROWS (m): | 3 |
| BOLT/m$^2$ | 0.24 |
| | |
| RESIN CAPSULE DIAMETER (m): | 19 |
| RESIN TYPES | SLOW & FAST |
| NUMBER OF RESIN CAPSULES: | 2 |
| | |
| BORD WIDTH (m): | 7 |
| PILLAR WIDTH (m): | 25 |
| DEPTH (m): | 178 |
| MINING HEIGHT (m): | 3.8 |
| SAFETY FACTOR: | 1.91 |

Cutting sequence

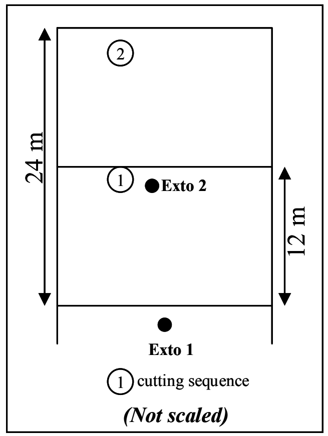



***Figure 4-16 Colliery 'E' Site 1***

## 4.9 Colliery 'F'

One site was monitored at Colliery ‘F’. The colliery is situated in the Vereeniging Coalfield and mining was conducted on the No 2B Seam.

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### 4.9.1 Colliery 'F' Site 1

Site 1 was a four-roadway, primary bord and pillar production section. The sonic probe monitoring holes were drilled in the one-right roadway. An onboard CM together with shuttle cars was used in the section. The installation and performance of support was also excellent.

The CM experiment sequence was used at this site. After the installation of the first hole was completed at the face, the initial reading was taken. The face was then advanced by 5.0 m with a 3.5 m drum width and the second hole drilled and instrumented with sonic probe anchors. Readings were taken from both holes. The second lift was then mined up to 5.2 m bord width, and readings were taken again from both holes. The experiment was completed by cutting a further 5.0 m. After this stage of the experiment, the area was supported due to a slip running across the roadway next to the second monitoring hole. A further reading was taken when the face advance was 60 m. The results obtained from both holes during the experiment in Colliery 'F' are presented in Figure 4-17. The summary of site performance is given in Table 4-14.

Similar to Colliery 'A' Site 1, Test 1 and 2, the water aquifer limited the sonic probe hole lengths. Therefore, approximately 3.0 m long sonic probe holes were drilled and monitored. The results showed that both holes were stable throughout the experiment.

***Table 4-14 Site performance Colliery 'F' Site 1***

| | | | |
|---|---|---|---|
| Coalfield: | Vereeniging | Seam: | No 2 B |
| Site: | One | Positions: | Roadway |
| Road widths: | 5.2 m | Pillar widths: | 44.1 m |
| Mining height: | 2.6 m | Depth: | 44.1 m |
| Mining method: | CM and shuttle cars | | |
| Cut-out distance | 10 m | | |
| Roof strata: | 1.41 m thick coal/shale layers overlain by 1.1 m thick coal/shale | | |

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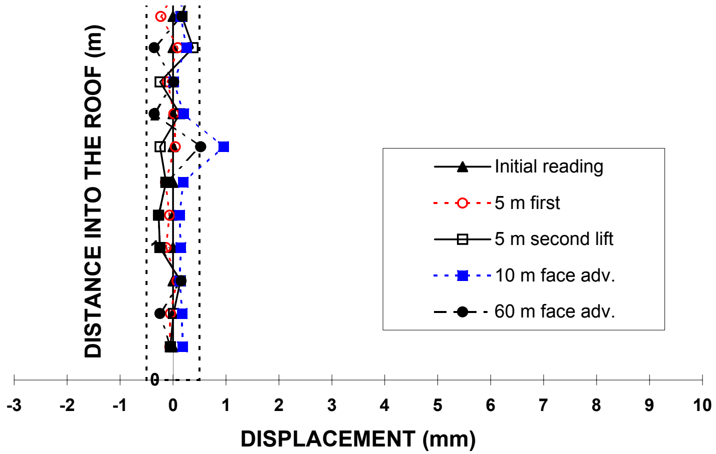
Support: 1.8 m x 20 mm Re-bar, partial column resin in 25 mm hole.
Support density 0.77 bolt/m$^2$.

Performance: Both holes were stable throughout the experiment, and no displacements were recorded in either hole.

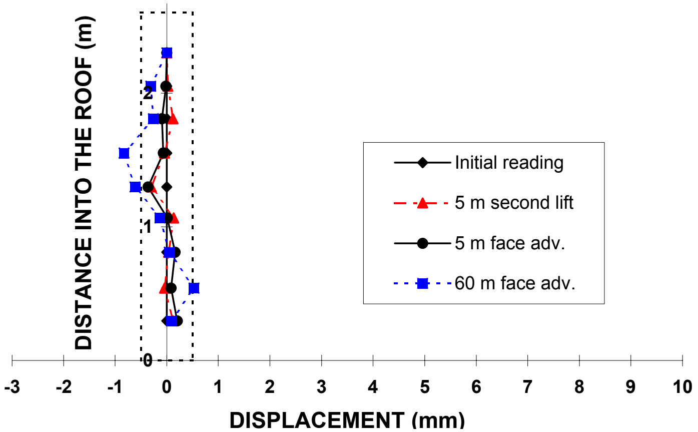
VEREENIGING COALFIELD
No 2B Seam

COLLIERY “F” - SITE 1
(CUT OUT DISTANCE 10 m)

HOLE 1




HOLE 2




Immediate roof lithology (not scaled)

| DEPTH INTO THE ROOF (m) | SECTION | WIDTH (m) | RECORD OF STRATA |
|---|---|---|---|
| 1.41 | | 1.1 | COAL - SHALE |
| 1.28 | | 0.13 | DULL COAL |
| 0.89 | | 0.39 | COAL - SHALE |
| 0.54 | | 0.35 | DULL COAL |
| 0 | | 0.54 | COAL - SHALE |

Site performance

| | |
|---|---|
| BOLT TYPE: | RE-BAR |
| BOLT DIAMETER (mm): | 20 |
| HOLE DIAMETER (mm): | 24 |
| BOLT LENGTH (mm): | 1800 |
| HOLE LENGTH (mm): | 1830 |
| NUMBER OF BOLTS IN A ROW: | 6 |
| DISTANCE BETWEEN THE ROWS (m): | 1.5 |
| BOLT/m$^2$ | 0.77 |
| | |
| RESIN CAPSULE DIAMETER (m): | 19 |
| RESIN TYPES | SLOW & FAST |
| NUMBER OF RESIN CAPSULES: | 3 |
| | |
| BORD WIDTH (m): | 5.2 |
| PILLAR WIDTH (m): | 28.8 |
| DEPTH (m): | 44.1 |
| MINING HEIGHT (m): | 2.6 |
| SAFETY FACTOR: | 12.31 |

Cutting sequence

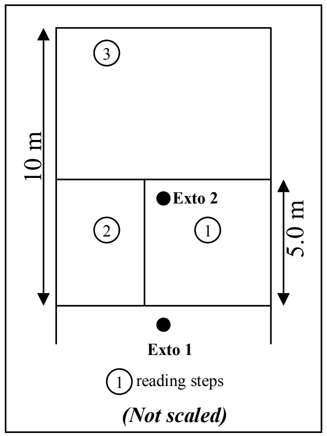



***Figure 4-17 Colliery 'F' Site 1***

## 4.10 Analysis of underground monitoring results

In the initial phase of the underground experiments, it was intended that in all the sites the cut-out distance would be extended to a point where the roof would fail and the critical deformations could be determined. However, discussions with mining personnel highlighted the fact that this was not possible, because all the experiments were to take place in normal production sections where roof failure could not be tolerated. Therefore, in 9 of 13 sites, only the standard cut-out distances of individual sections were monitored. In the remaining four sites (Colliery 'B' Sites 1 and 2, Colliery 'C' Site 3, and Colliery 'E' Site 1) extensive cut-out distances of 18 m to 31 m were monitored. The results, however, showed that the roof deformations were as small as for the significantly shorter cut-out distances. For example, in Colliery 'B' Site 2, the 31 m cut-out distance showed no dilation in the roof. This of course could be due to installing the sonic probe holes late or due to the very competent coal roof.

The results obtained from both the No 1 and No 2 sonic probe monitoring holes from all the monitoring sites are given in Table 4-15 and Table 4-16. The results did not show any obvious correlation between the maximum dilation and other variables. In fact this could be expected as there are many parameters, which can affect the roof performance. These include: the support density, roof lithology, bord width, stress changes in the roof as well as cut-out distance. While these parameters may affect the roof on an individual basis, it is shown that generally more than one of the above factors will have an influence on roof behaviour. Therefore, in order to obtain repeatability of experiment results, more than one experiment is required from each site, which was impossible to achieve in this study.

The relationships between the maximum dilation and the support density, roof lithology, bord width and cut-out distance are shown from Figure 4-18 to Figure 4-21. It can be seen from these figures that there is no obvious correlation between roof dilation and the other variables. Consequently, the data does not show meaningful relationships between these parameters. There is thus no indication of the dependence of roof stability on cut-out distance in the range of cut-out distances that was investigated.

There was, however, one significant correlation observed, and this was the relationship between the position where separation occurred and the thicknesses of lithological units. This is shown in Figure 4-22. The immediate roof thicknesses were obtained from the borehole logs and the position of the separation from the sonic probe monitoring observations. This figure confirms that the position of dilation or separation in the roof agreed with the position of likely partings or change in rock type in the roof. This finding indicates that as a fundamental analytical tool, beam theory may be used to estimate the expected roof behaviour.

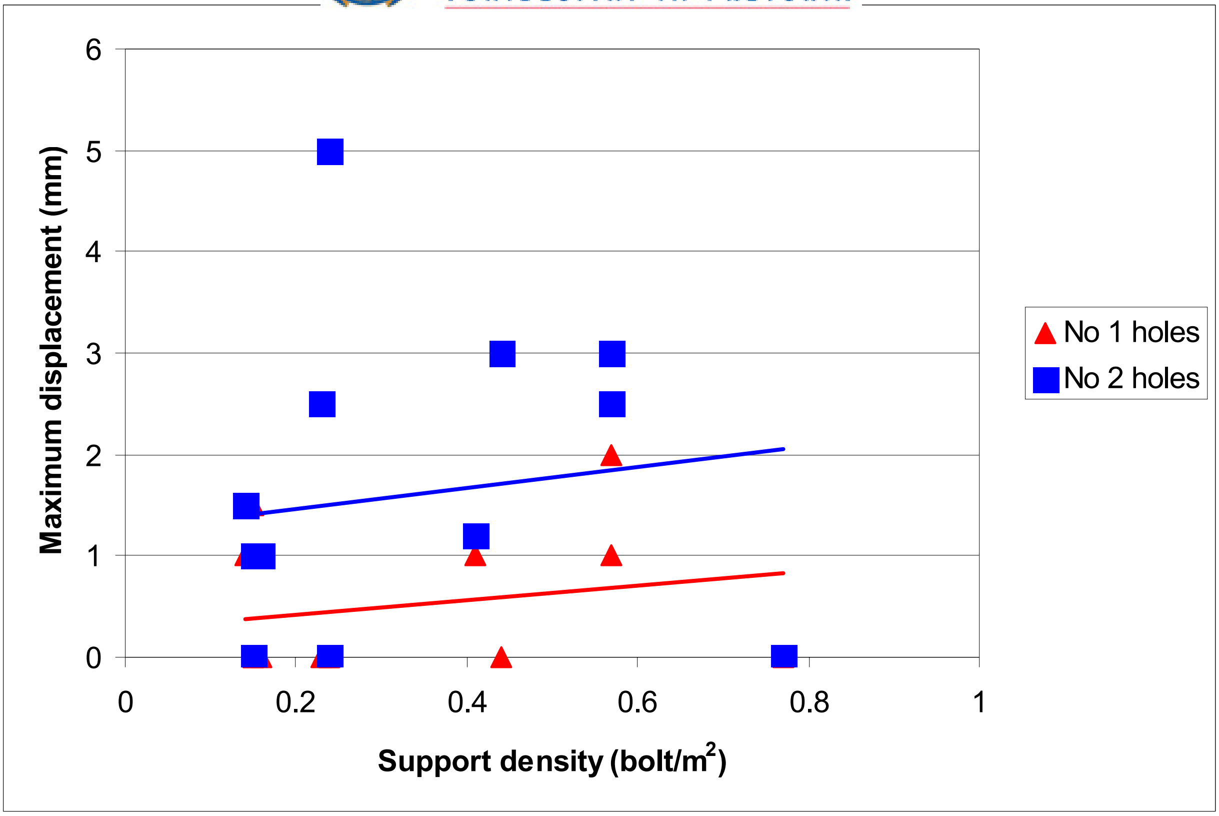



*Figure 4-18 The relationship between the support density and total displacement*

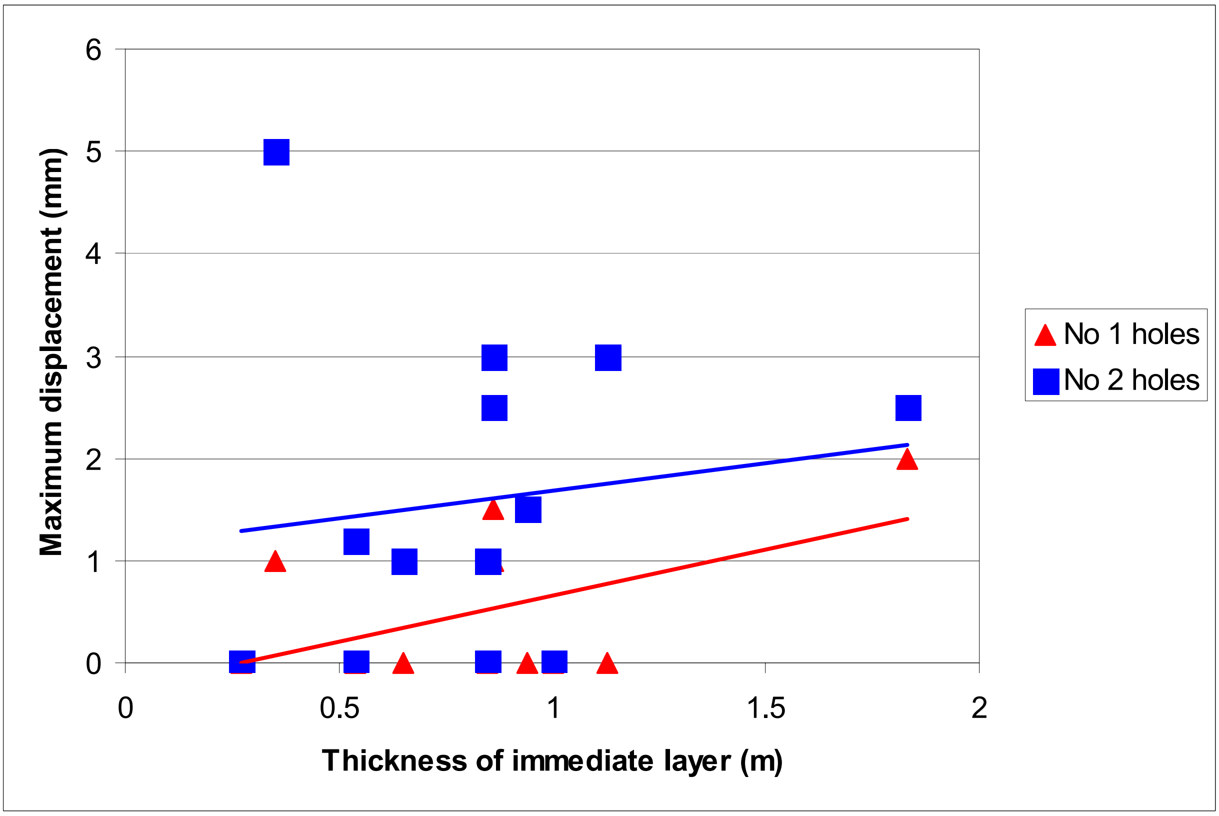



*Figure 4-19 The relationship between the thickness of the immediate layer and total displacement*

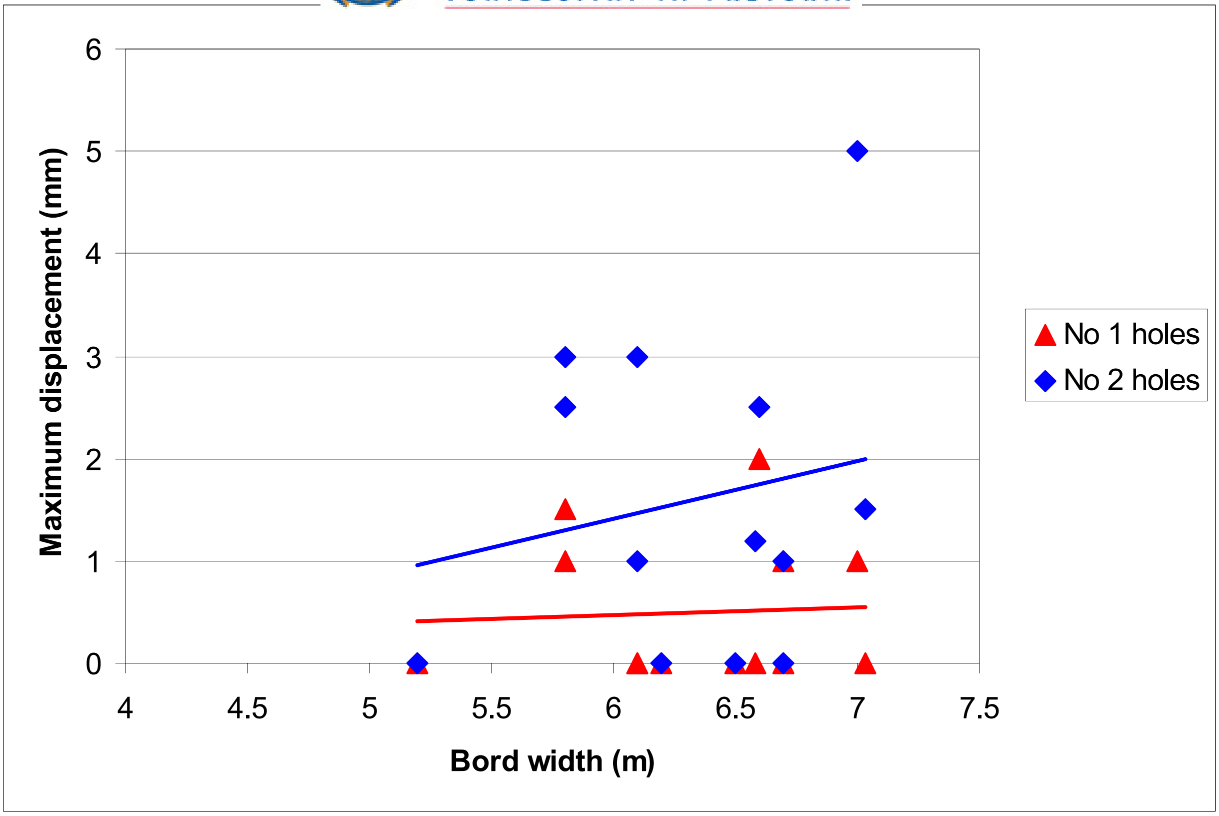



***Figure 4-20 The relationship between the bord width and total displacement***

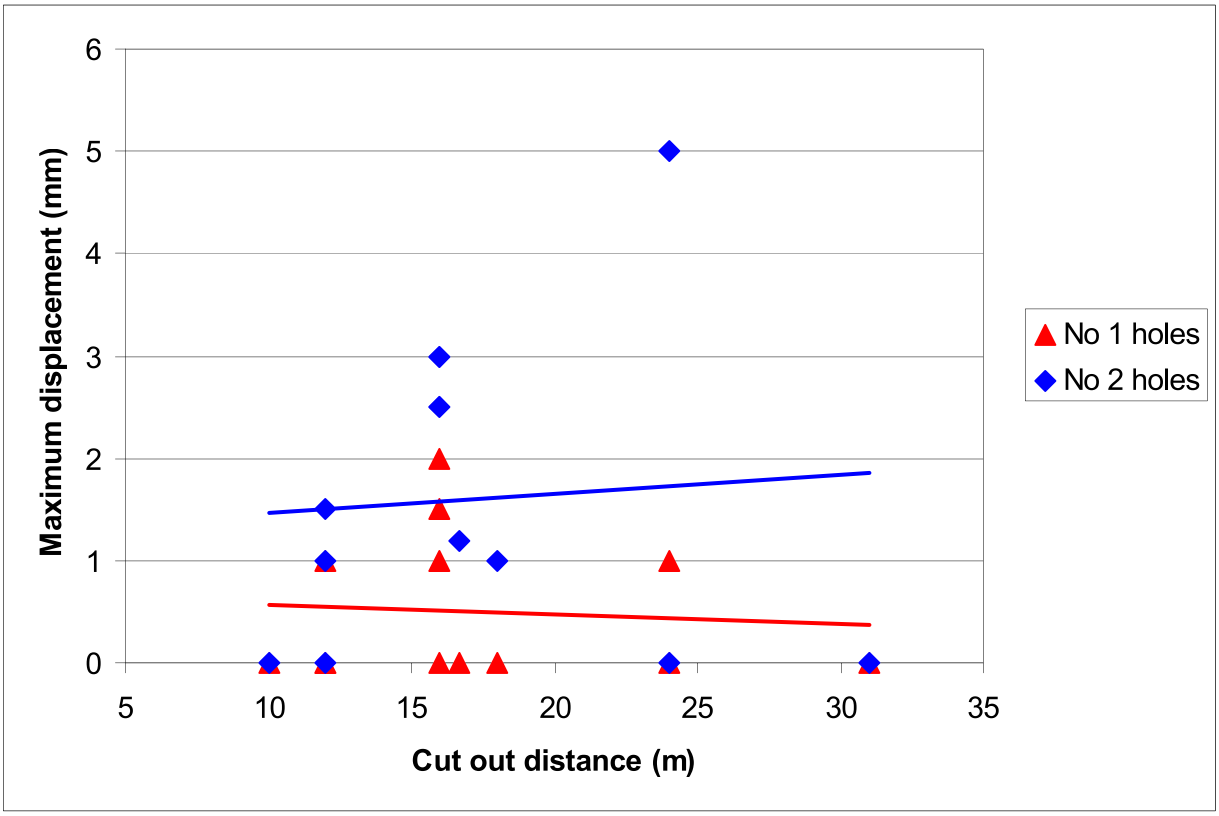



***Figure 4-21 The relationship between the cut-out distance and total displacement***

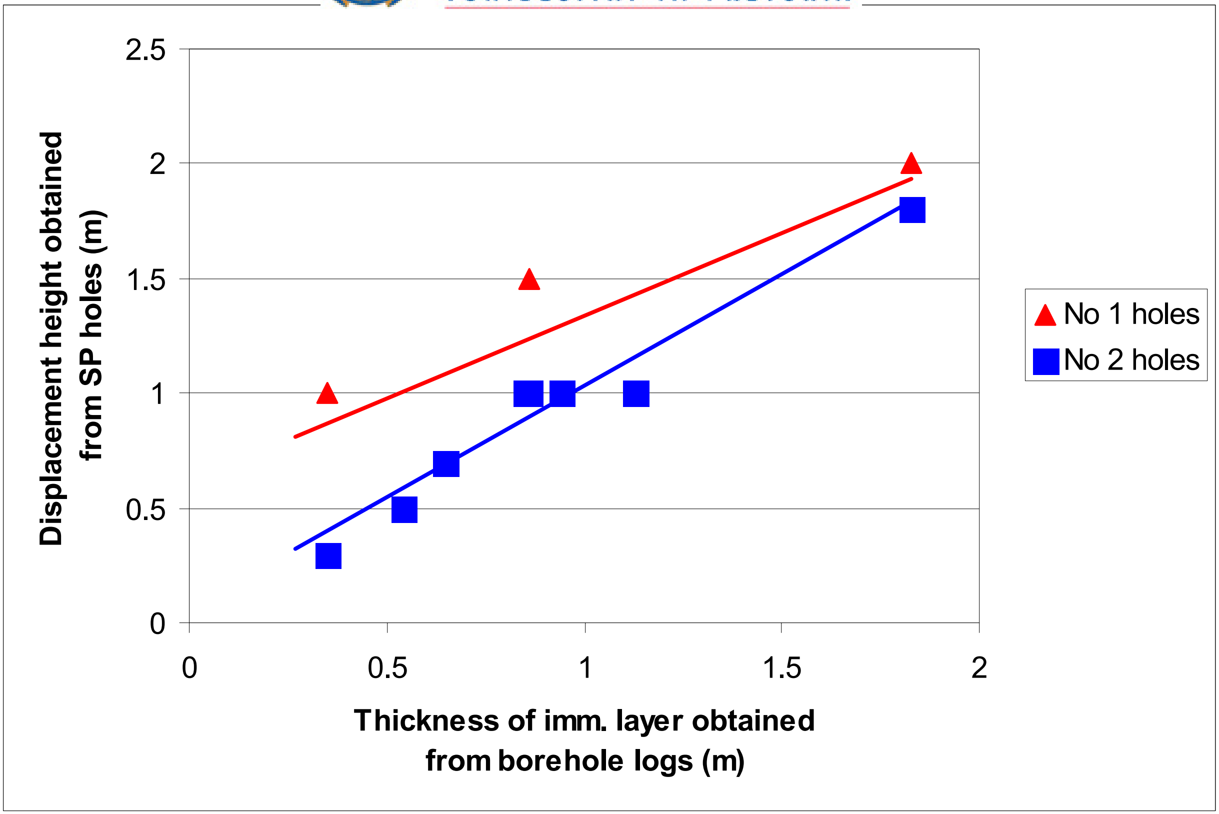



***Figure 4-22 The relationship between the thickness of the immediate layer obtained from the borehole logs and height of the displacement obtained from underground sites where some degree of dilation was recorded***

The time it took for deformation to occur was similar in all the sections. In all the sections the greater portion of the displacements took place during and immediately after mining took place at the face. On average 69 per cent of the maximum dilation measured took place during the face advance, and a further 11 per cent during the following 48-hours unsupported stand-up time. This figure is based on data from both No 1 and No 2 sonic probe monitoring holes where some degree of dilation was recorded in the roof.

This indicates that on average, 69 per cent of the deformation takes place immediately after the mining takes place at the face, before the support has been installed. If the support installation is delayed by 48-hours, this percentage rises to 80 per cent. Although the percentage increase after a 48-hour unsupported stand-up time is not significant, it may change the roof behaviour from elastic to plastic, due to the weathering and development of micro fractures.

Two attempts were made to identify the effect of roof bolting and tensioning of roof bolts. This was done in Colliery 'B', Site 3 and Colliery 'D', Site 1. However since no roof movement was recorded in holes No 1 and 2, in Colliery 'B', and although some initial displacement took place above the bolt horizon in holes No 1 and No 3 in Colliery 'D', no displacement was recorded within the bolt horizon. Hole No 2 in Colliery D presented an ideal site to attempt to determine the effect of the installation of the roof bolts. The installation of the bolts appeared to have little if any effect on the bed separation already evident within the bolted interval as can be seen in Figure 4-15. However, as this information came from only one monitoring hole, it cannot be concluded that this is typical and that the installation of pre-tensioned roof bolts has no remedial effects on pre-existing openings within the bolted interval.

Although the effect of the installation of pre-tensioned roof bolts was specifically monitored at only one site, in general in the monitoring holes, where the displacements occurred within the roof bolt horizon, there was no evidence that the installation of the bolts partially closed pre-existing openings within the roof strata. Whether the roof stability would be improved by reversing some of the relaxation that takes place prior to the installation of the roof bolts is open to debate. This requires a further investigation.

Based on the good correlation that was found between the thicknesses of the nether roof units and the positions where dilation (or separation) occurred, it was concluded that the roof displayed characteristics of discrete plate behaviour. This behaviour was investigated further by comparing the measured roof deflections with that which is predicted by a conventional gravity loaded beam theory. As the length of the roadways exceeded twice their width in all cases, the valid simplification of using beam formulae was introduced.

The deflection of the roof was predicted using the following equation:

$$\eta = \frac{\gamma B^4}{32Et^2} \qquad [4\text{-}5]$$

where $\eta$ = Maximum deflection (m),

$\gamma$ = unit load *($\rho g$)*,

$E$ = Modulus of Elasticity,

$t$ = thickness of layer and

$B$ = bord width.

Where appropriate, allowance was made for additional load resulting from softer layers overlying stiffer ones. Where laminated layers consisted of materials of different stiffnesses, a weighted average stiffness was used in the calculations.

The comparison between calculated and measured deflections is shown in Figure 4-23. Although the correlation coefficient of the linear line is low (53 per cent), the good correlation is immediately apparent, as is the fact that the magnitudes of the displacements are in the same range. It can therefore be concluded that the gravity loaded beam analogy is valid for predicting roof behaviour in the study area. The correlation coefficient is affected by mainly two cases where very little deflection was predicted. This may have been caused by incorrect loading assumptions.

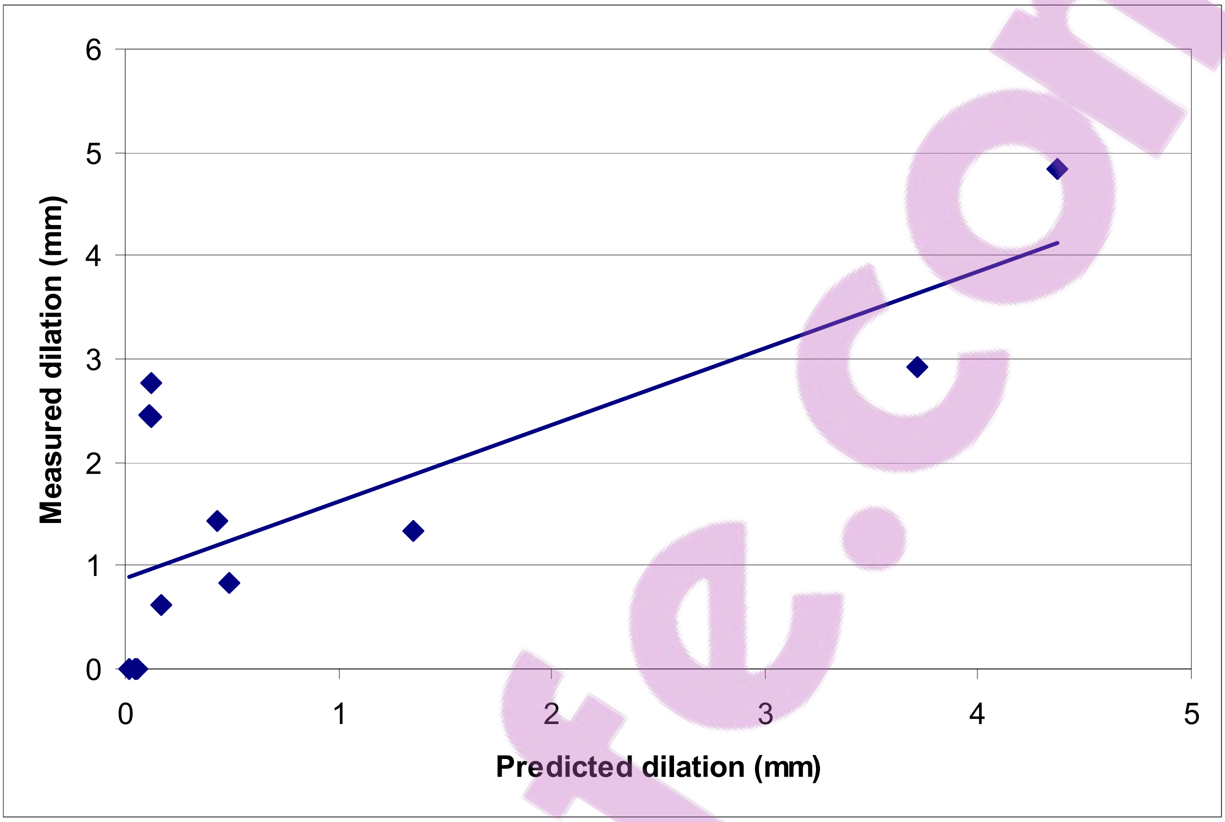



***Figure 4-23 Relationship between measured and predicted dilation***

The simplification of using beam theory rests on the assumption that once the length of a plate exceeds twice its width, further increases in length will not result in meaningful additional deflection. The good correlation between the predicted beam deflections and the measured plate deflections confirm that further increases in length (i.e. cut-out distance) will not result in meaningful additional roof deflection. This agreed with the underground observations, where it was indicated that stability was reached very soon, i.e. predominantly during mining of the limited cut-out distance.



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***Table 4-15 Summary results obtained from No 1 sonic probe monitoring holes***

| Colliery | Site | Max. height of dilation (m) | Support density (bolt/m^2) | Bord width (m) | Cut-out distance (m) | Total displ. (mm) | Immediate roof lithology | Thickness of imm. roof (m) |
|---|---|---|---|---|---|---|---|---|
| A | 1, Test 1 | 0.8 | 0.57 | 5.8 | 16 | 1.0 | Grit/grit coal laminae | 0.86 |
| A | 1, Test 2 | 1.5 | 0.57 | 5.8 | 16 | 1.5 | Grit/grit coal laminae | 0.86 |
| A | 2 | 0 | 0.41 | 6.58 | 16.7 | 0.2 | Sandstone | 0.54 |
| A | 3 | 0 | 0.44 | 6.1 | 16 | 0.2 | Sandstone and grit | 1.13 |
| B | 1 | 0 | 0.15 | 6.5 | 24 | 0.2 | Coal | 1 |
| B | 2 | 0 | 0.15 | 6.7 | 31 | 0.2 | Coal | 0.85 |
| B | 3 | 0 | 0.24 | 6.2 | 12 | 0.2 | Sandstone | 0.27 |
| C | 1 | 0 | 0.14 | 7.03 | 12 | 0.2 | Mudstone/coal | 0.94 |
| C | 2 | 0.5 | 0.15 | 6.7 | 12 | 1.0 | Mudstone/coal/sandstone | 0.85 |
| C | 3 | 0 | 0.16 | 6.1 | 18 | 0.2 | Shale/coal | 0.65 |
| D | 1 | 1.8 | 0.23 | 6.6 | 16 | 2.0 | Coal/shaly coal | 1.83 |
| E | 1 | 0.5 | 0.24 | 7 | 24 | 1.0 | Coal/sandstone | 0.35 |
| F | 1 | 0 | 0.77 | 5.2 | 10 | 0.2 | Coal/shale | 0.54 |

***Table 4-16 Summary results obtained from No 2 sonic probe monitoring holes***

| Colliery | Site | Max. height of dilation (m) | Support density (bolt/m^2) | Bord width (m) | Cut-out distance (m) | Total displ. (mm) | Immediate roof lithology | Thickness of imm. roof (m) |
|---|---|---|---|---|---|---|---|---|
| A | 1, Test 1 | 1 | 0.57 | 5.8 | 16 | 3.0 | Grit/grit coal laminae | 0.86 |
| A | 1, Test 2 | 1 | 0.57 | 5.8 | 16 | 2.5 | Grit/grit coal laminae | 0.86 |
| A | 2 | 0.5 | 0.41 | 6.58 | 16.7 | 1.2 | Sandstone | 0.54 |
| A | 3 | 1 | 0.44 | 6.1 | 16 | 3.0 | Sandstone and grit | 1.13 |
| B | 1 | 0 | 0.15 | 6.5 | 24 | 0.2 | Coal | 1 |
| B | 2 | 0 | 0.15 | 6.7 | 31 | 0.2 | Coal | 0.85 |
| B | 3 | 0 | 0.24 | 6.2 | 12 | 0.2 | Sandstone | 0.27 |
| C | 1 | 1 | 0.14 | 7.03 | 12 | 1.5 | Mudstone/coal | 0.94 |
| C | 2 | 1 | 0.15 | 6.7 | 12 | 1.0 | Mudstone/coal/sandstone | 0.85 |
| C | 3 | 0.7 | 0.16 | 6.1 | 18 | 1.0 | Shale/coal | 0.65 |
| D | 1 | 1.8 | 0.23 | 6.6 | 16 | 2.5 | Coal/shaly coal | 1.83 |
| E | 1 | 0.3 | 0.24 | 7 | 24 | 5.0 | Coal/sandstone | 0.35 |
| F | 1 | 0 | 0.77 | 5.2 | 10 | 0.2 | Coal/shale | 0.54 |

## 4.11 Investigation of trends using numerical modelling

As mentioned earlier, the behaviour of the roof is a function of many variables. These include the stress environment, roof lithology and strength of roof materials, bord width, etc. A further complication is that the variables govern the roof behaviour according to their combination with the others. There are a great number of different combinations, and although great care was taken to include the widest possible range of parameters in the study, it was clearly not possible to include all or even a sufficient number to derive all the answers experimentally.

However, important trends were identified. To investigate these trends further, a numerical modelling code was added to the study to conduct a comparative analysis. The three dimensional (3D) boundary element code Map3D was used in the analysis. The basic 3D model that was used is shown in Figure 4-24.

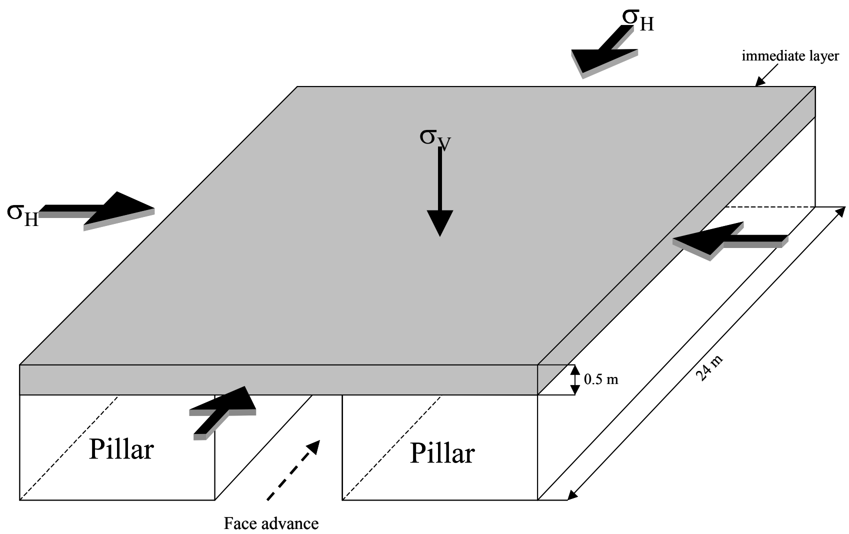


***Figure 4-24 MAP3D model that was used in the numerical modelling analysis***

The following material properties were used in all models:

***Table 4-17 Input parameters used in numerical modelling***

| Material | Elastic modulus (GPa) | Passion's Ratio | Density (kg/m$^3$) |
|---|---|---|---|
| Host material | 15.0 | 0.20 | 2500 |
| Immediate layer | 8.0 | 0.25 | 2500 |
| Coal | 4.0 | 0.28 | 1600 |

The first variable that was investigated was bord width, Figure 4-25. The trend is clear – the greater the bord width, the greater the roof deflection, as would be expected.

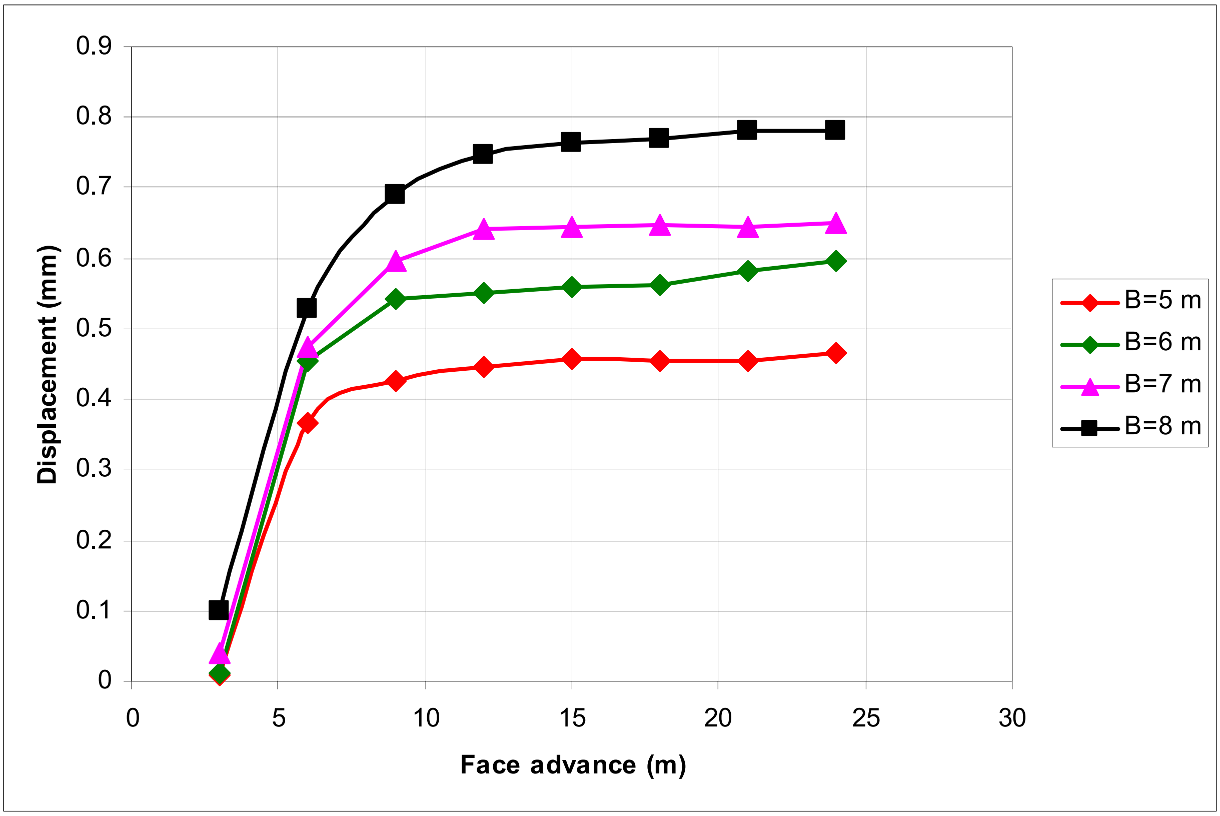


***Figure 4-25 Effect of bord with on dilation***

The effect of horizontal stress was investigated next. A 0.5 m thick immediate layer overlain by sandstone overburden was modelled, and the maximum displacements at 6.0 m face advances were measured in the centre of the bord. While all the parameters were kept constant, the model was run with two k-ratios (ratio of horizontal stress to vertical stress) of 3.0 and 10. The results are shown in Figure 4-26. This figure indicates that increasing horizontal stress by a factor of 3.3 increased the deformations in the roof by a factor of 1.3 at 6.0 m advance and by smaller proportions at greater advances. That increasing the horizontal stress did not accelerate the roof deflection is confirmed by observations at the three sites where horizontal stress was regarded as a problem. At those sites, the roof deflection did not differ significantly from that at any other site. However, the magnitude of the stresses at the sites was not known.

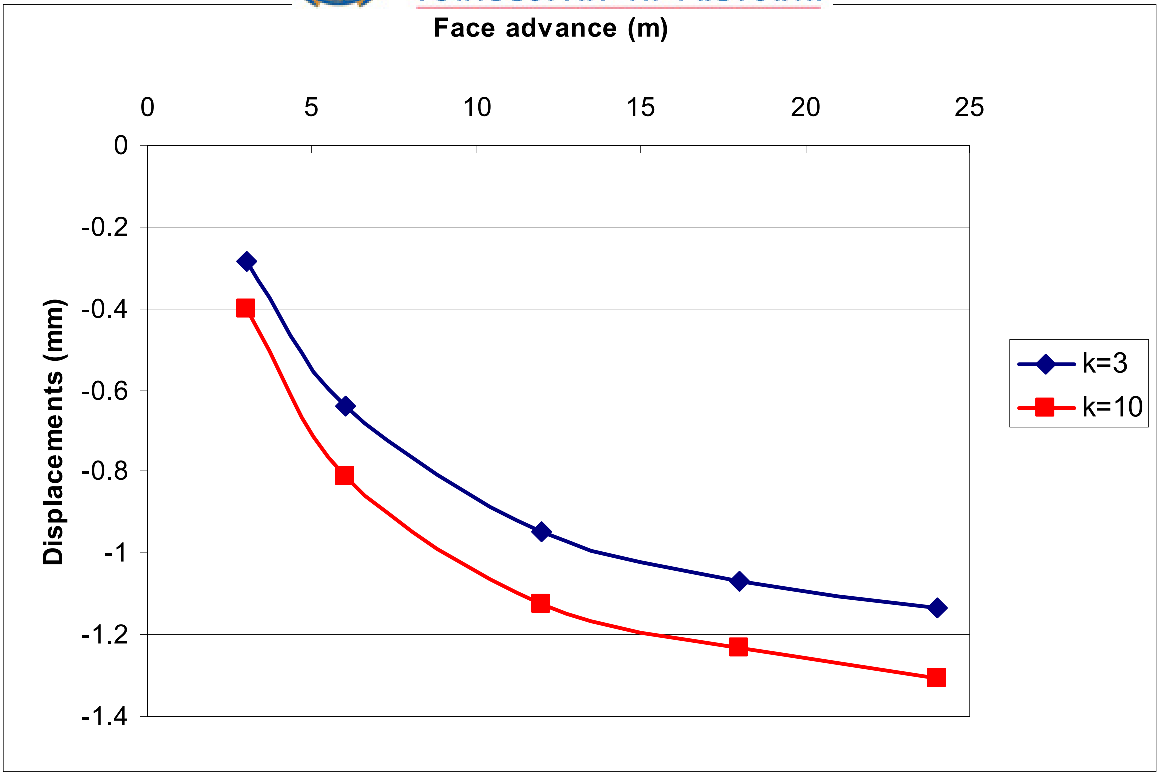


***Figure 4-26    Effect of k-ratio on roof deformations***

Note that this observation should not be read as implying that elevated levels of horizontal stress are irrelevant. It merely means that if the stress is not high enough to result in failure of the roof material, it will not dramatically increase the deflection of the roof (until of course it reaches stress levels that are sufficient to induce buckling of the roof beam, when sudden failure can be expected).

The effect of the thickness of the immediate layer was also investigated using the same model. In this model the k-ratio was kept constant at 3.0, and 0.25 m, 0.5 m and 1.0 m thick, immediate layers were modelled. The results indicated that decreasing the thickness of the immediate layer increases the displacements in the roof, Figure 4-27. This trend was also apparent in the monitoring, indicated by relatively good correlation between measured deflections and the beam predictions.

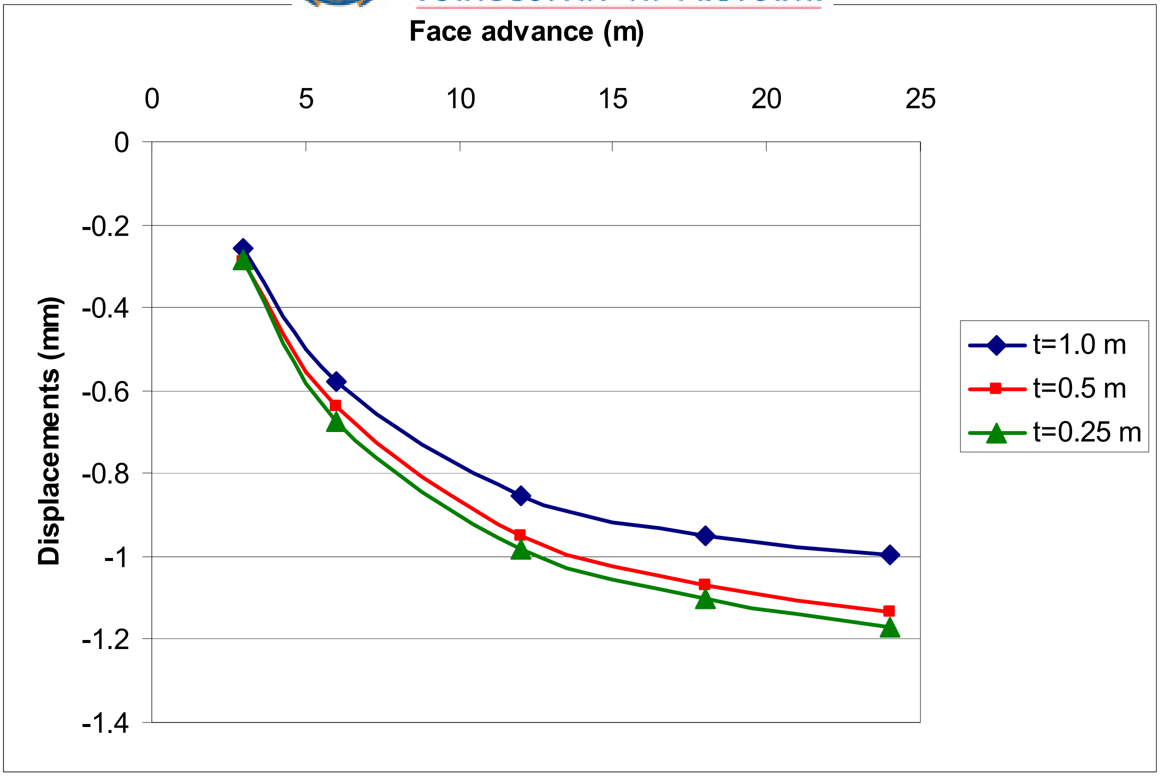


***Figure 4-27 Effect of the thickness of the immediate layer on roof deformations***

Other critical parameters in determining the deformability of the roof are the elastic properties of the immediate layer. The same model as given in Figure 4-24 was used in the analysis. While the k-ratio was kept constant (k = 3), the material properties of the immediate layer were changed. In the first model, the Elastic Modulus and Poisson's ratio of the immediate layer were taken as 10 GPa and 0.18 respectively. In the second model less stiff material was used. The Elastic Modulus and Poisson's ratio in the second model were 2.5 GPa and 0.22, respectively. The results obtained from the modelling are shown in Figure 4-28. As can be seen from this figure, the properties of the immediate layer have a major effect on the deformations in the roof.

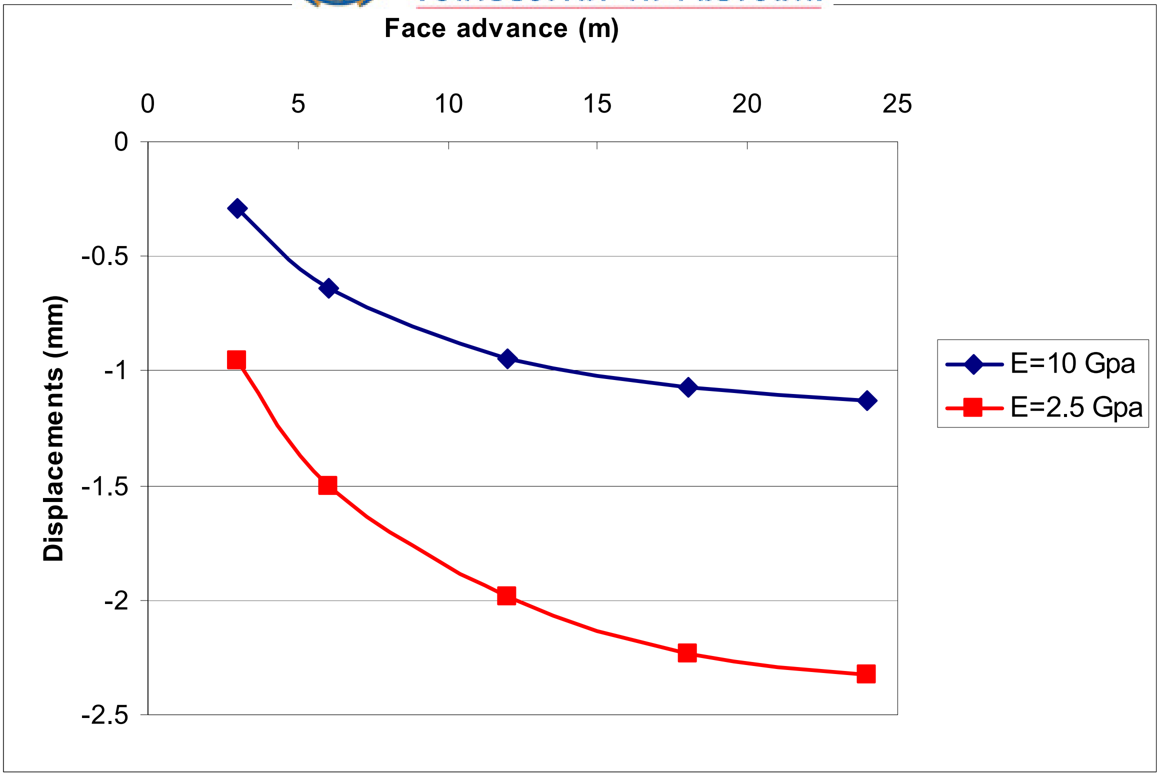


***Figure 4-28  Effect of the strength of the immediate layer on roof deformations***

The conclusion drawn from the numerical modelling is that the trends observed by underground monitoring are confirmed. It is clear that roof behaviour is the result of a complex combination of several variables. The benefit of modelling is also illustrated. It is the only practical way in which selected parameters can be varied while the others are kept constant in order to isolate the contribution of the different parameters to the overall observed behaviour.

The most important conclusion is that face advance only plays an important role during the initial advance, until the advance equals twice the bord width. Thereafter the additional roof deflection is not significant. However, bord width is important from the beginning and the effect never diminishes. The same can be concluded for the thicknesses of the lithological units and their properties.

The underground observations represent points on the trend curves of overall behaviour. There were insufficient data points to determine trends for each of the variables in isolation, but with the aid of the models it was seen that the observations fitted the patterns exhibited by the models.

## 4.12 Conclusions

The literature survey yielded little in the form of directly applicable research. It appeared that little work on determining roof failure per se as a function of cut-out distances has been done elsewhere. The limitations on cut-out distances were mainly due to other issues, like preventing underground workers being under unsupported roof and methane and dust control. Recent work done by researchers in the USA seems to indicate that extending the cut-out distance in the USA had little effect on roof stability, mainly because operators tended to reduce the cut-out distance under adverse roof conditions and only extend it if roof conditions were good.

The ideal research methodology from a scientific viewpoint would have been to advance unsupported faces until failure occurred. If this could be done under a sufficient number of different situations, it would have been possible to provide direct answers for different situations. However, it was not possible to do that without exposing people to considerable risk. The next best was to monitor the universally accepted precursor to roof failure, which is roof deflection, under a range of different situations. This was done under the widest possible range within the constraints of time and funds, but it was still found that there were too many combinations of the variables that determine the roof deflection to derive complete answers.

The measurements were then complemented by numerical monitoring, which affords the possibility to vary only certain parameters and keep the rest constant. It was then found that the underground observations fitted the patterns derived from the models and consequently there is a high level of confidence in the final conclusions.

The most important conclusion from this investigation was that once the face had advanced to a distance equal to twice the bord width, there was insignificant additional roof deflection with further face advance. This conclusion was confirmed by numerical modelling and is in line with the analytical beam solutions. For typical South African conditions, with bord widths in the range of 5.5 m to 7.2 m, the implication is that roof stability would not be adversely affected by advancing further than 11 m to 14 m. Majority of all of the total roof deflection that would take place, would occur during the first 11 m to 14 m of development. Therefore, if it is intended to limit roof deflection by restricting the cut-out distance, the cut-out distance would have to be limited to less than the bord width. During the investigation, it was observed that where adverse roof conditions existed, this was in fact done by underground personnel.

With regard to the effects of time on roof deflection, it could only be studied for the initial period of 48-hours following roof exposure. The reason for this was operational, as leaving faces for longer periods would have had an adverse effect on production and the sequence of mining.

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The instrumentation was usually done on Friday afternoons, preceding weekends during which faces would not be mined. It was found that the roof continued to deflect during that period, but that the amounts of deflection were not significant. However, it is still deemed necessary to support a roof as soon as possible, as even minute fractures resulting from the additional deflection may change the roof behaviour and eventually result in failure.

Results from one sonic probe monitoring hole showed that roof bolting had minimal remedial effect on roof deformations. Although the effect of roof bolting was specifically monitored by only one sonic probe monitoring extensometer, in general, the results showed that in none of the monitoring holes where roof displacements were recorded, was there any evidence of the roof being lifted due to installation of pre-tensioned roof bolts. This indicates that the roof bolt tensioning was not sufficient to close the pre-existing openings within the roof strata, where roof displacements were recorded. However, as indicated by the differences in the maximum displacements between the No 1 and No 2 holes, it may be concluded that roof bolting prevented further deterioration from taking place. In all the cases the displacements recorded by the No 1 holes (drilled next to the previously installed bolts) were less than those recorded by the No 2 holes (drilled in the centres of the unsupported areas) during the same monitoring period.

It was found that the lithological composition of the roof strata played a major role in the amounts of deflection that were recorded. Bedding separation was seen to occur at the positions where different strata types joined. This implies that the roof behaved like a set of composite beams of different characteristics. It was then also found that the amounts of deflection corresponded with the deflection that would be expected from gravity loaded beams.

Within the limits of horizontal stress that were present in the study areas (three of the sites exhibited obvious signs of elevated horizontal stress), the stress appeared not to have had a noticeable effect on roof deflection. This was confirmed by the numerical modelling. It was concluded that as long as the magnitude of the stress is insufficient to result in failure of the roof, it does not contribute meaningfully to deflection.

The implication of this is that the dilation in the roof is determined by bord width and roof lithology rather than cut-out distance, once the cut-out distance exceeds twice the bord width.

This last conclusion is significant, as it offers the first possibility to predict roof deflection and consequently roof failure. The recommended process is as follows:

- Determine the thicknesses of the roof plates (or beams) by careful scrutiny of borehole logs.
- Calculate the maximum deflection for the desired road width using standard beam solutions.
- Calculate the induced beam stresses using the standard beam solutions.
- If failure is not predicted, the road width is confirmed.
- The cut-out distance should be determined by other considerations (ventilation requirements, etc), but at least it is known that there is little to be gained in terms of roof stability by restricting it to any distance that is greater than twice the bord width.

Roof deflection should then be monitored underground and the first warning sign should be where the amount of deflection exceeds the calculated amount, as that would indicate a change in conditions. Where that occurs, it would be prudent to reduce the cut-out distance, but even more so to reduce the road width.

Exemption from the 12 m restriction on cut-out distance may be obtained from the Principal Inspector provided that the mine can show that the risk to underground workers will not be adversely affected. This implies that a comprehensive risk assessment is required to obtain the exemption. The results of this investigation show that in general, the increased risk to roof instability due to extended cut-out distances is not a major factor and that the emphasis in the risk assessment should be on the other factors, namely the control of dust and methane and the probability of workers being under unsupported roof.

As with any matter relating to roof stability, it is recommended to base this type of exemption on a comprehensive hazard analysis. It is important to obtain a broad view, based on a general roof hazard plan that is required for other purposes as well.

The following steps are recommended for determining the effective cut-out distances for a given site:

1. General roof hazard plans should be drawn up for each section based on the borehole logs,
2. A detailed geotechnical analysis should be conducted. This analysis should include mapping of geological discontinuities, stress regime and roof lithology,
3. The characteristic behaviour of the roof should be determined for the range of conditions, such as change in the thickness of the immediate roof layer, stress regime and bord widths,
4. Once the bord width and support method are established from the above, the cut-out distance can be determined as well. The most important control parameter is

the bord width. If the bord width is chosen such as to result in deflection that is less than that resulting in failure using beam theory, there is little to be gained by restricting the cut-out distance.

5. With the previous steps in place, it remains to also stipulate a procedure that will prevent any person being under unsupported roof.
6. The support system that will be used in the section should also be monitored by continuing the monitoring after the installation of support. The critical factors in determining the support performance are the height of the instability into the roof, which determines the length of support, and the separations within the bolt horizon, which determine the stiffness of the support.
7. Once the cut-out distance is determined with regard to ground control, it should be checked against the ventilation and risk assessment plans.

The study area included one site where there was a high incidence of jointing, but in that area the effects of the jointing did not materialise in the measurements, most probably due to "experimental gremlins." The irony is that the roadways next to the one where the instrumentation was done suffered severe damage and the cut-out distances in those were reduced substantially by the operational crews. However, in the instrumented roadway, no damage occurred and the roof deflection was minimal.

Finally, logic dictates that the longer the cut-out distance, the higher the probability of encountering unexpected jointing with its accompanying negative effects on roof stability. This may be countered by instituting measures that will prevent personnel being under unsupported roof.

# Chapter 5.0

# Evaluation of geotechnical classification techniques to design coal mine roofs

## 5.1 Introduction

Rock mass classification systems have constituted an integral part of empirical mine design for over 100 years, Ritter (1879). The use of such systems can be either implicit or explicit. They are traditionally used to group areas of similar geotechnical characteristics, to provide guidelines of stability performance and to select appropriate support. In more recent years, classification systems have often been used in tandem with analytical and numerical tools. There has been an increase of work linking classification indexes to material properties such as modulus of elasticity, the $m$ and $s$ parameters in the Hoek and Brown (1988) failure criterion, etc. These values are then used as input parameters for numerical models. Consequently, the importance of application of rock mass characterization methods has increased over time. The primary objective of all classification systems is to quantify the intrinsic properties of the rock mass based on past experience. The second objective is to investigate how external loading conditions acting on a rock mass influence its behaviour. An understanding of these processes can lead to the successful prediction of rock mass behaviour for different conditions.

The earliest reference to the use of rock mass classification for the design of tunnel support is by Terzaghi (1946) in which the rock loads, carried by steel sets, are estimated on the basis of a descriptive classification. Since Terzaghi (1946), many rock mass classification systems have been proposed, the most important of which are as follows:

- Lauffer (1958)
- Deere (1970): Rock Quality Designation, RQD
- Wickham et al. (1972): Rock Structure Rating (RSR – Concept)
- Bieniawski (1973): Geomechanics Classification, Rock Mass Rating
- Barton et al. (1974): Q- System
- Buddery and Oldroyd (1992): Impact splitting Test
- Molinda and Mark (1994): Coal Mine Roof Rating

Most of the multi-parameter classification schemes by Wickham et al. (1972), Bieniawski (1973, 1989) and Barton et al. (1974) were developed from civil engineering case histories in which

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most of the components of the engineering geological character of the rock mass were included. Studies of these systems have shown that their main application is for both hard and soft jointed rock masses. Several classification systems have been developed and modified for underground coal mining. Many mines locally and abroad have been using locally developed classification systems to determine the roof qualities and support systems that are in most cases not well documented and are restricted to the developer of such systems or the mine on which the system was developed. Furthermore, these systems cannot be compared with one another or results converted to an equivalent rating in another mine. In this Chapter, the application of CMRR in South African collieries is reviewed and evaluated against locally used impact splitting test developed by Oldroyd and Buddery (1992). The aim of this assessment is to evaluate rating system that is most appropriate for South African coal mines to design roof support systems.

Several authors in the past summarised the widely used rock mass rating systems, which are utilised in Civil Engineering tunnelling and in gold and platinum hard rock mines. These summaries can be found in the following references:

- Hoek, (2007)
- Swart (2005)
- Guler et al. (1998)
- Singh and Goel (1992)
- Milne et al (1998)
- Milne (1988)

## 5.2 Coal Mine Roof Rating (CMRR)

Molinda and Mark (1994) have developed the Coal Mine Roof Rating (CMRR) classification system to quantify descriptive geological information for use in coal mine design and roof support selection. This system results from years of geologic ground control research in longwall mines in the United States. The CMRR weights the geotechnical factors that determine roof competence, and combines them into a single rating on a scale from 0-100. The characteristics of the CMRR are that it:

- Focuses on the characteristics of bedding planes, slickensides, and other discontinuities that weaken the fabric of sedimentary coal measure rock.
- Applies to all U.S. coalfields, and allows a meaningful comparison of structural competence even where lithologies are quite different.

- Concentrates on the bolted interval and its ability to form a stable mine structure.
- Provides a methodology for geotechnical data collection.

The principle behind the CMRR system is to evaluate the geotechnical characteristics of the mine roof instead of the geological description. CMRR emphasizes structurally weak or strong units instead of geologic divisions. The structure of the system is similar to RMR (Bieniawski, 1973) system in that the important roof parameters are identified, their influence on roof strength is quantified and the final rating is calculated from the combination of all the parameters. Figure 5-1 shows the parameters that compose the CMRR system. The system is also designed such that the final rating/unsupported span/stand-up time relationship is comparable to that of the RMR. However, the CMRR is intended to be a universal system for coal mining and to initially exclude time-consuming and expensive laboratory analyses. Later, Molinda and Mark (1999) documented a revised approach that takes into consideration the Point Load Test.

An important attribute of the CMRR is its ability to rate the strength of bedded rocks in general, and of shales and other clay-rich rocks in particular. Layered rocks are generally much weaker when loaded parallel to bedding, and the CMRR addresses both the degree of layering and the strength of the bedding planes. In addition, the CMRR has been modified by Molinda and Mark (1999) to retain its ability to identify those rocks that are most susceptible to horizontal stresses.

Data gathering for the system relies only on observation and simple contact tests using a ball peen hammer, a 9 cm mason chisel, a tape measure and sample bags. All the data is recorded in a designed data sheet that is used to calculate the final rating. The calculation is based on rating the exposed roof that is divided into structural units. Each unit is rated individually, mainly on an evaluation of the discontinuities and their characteristics. Next, the CMRR is determined for the mine roof as a whole. The ratings of the units within the bolted interval are first combined into a thickness-weighted average. Then a series of roof adjustment factors are applied with the most important being that of the strong bed. It was found by the developers of the system that the structural competence of a bolted mine roof is largely determined by its competent member. All the parameters are then combined to calculate the final CMRR.

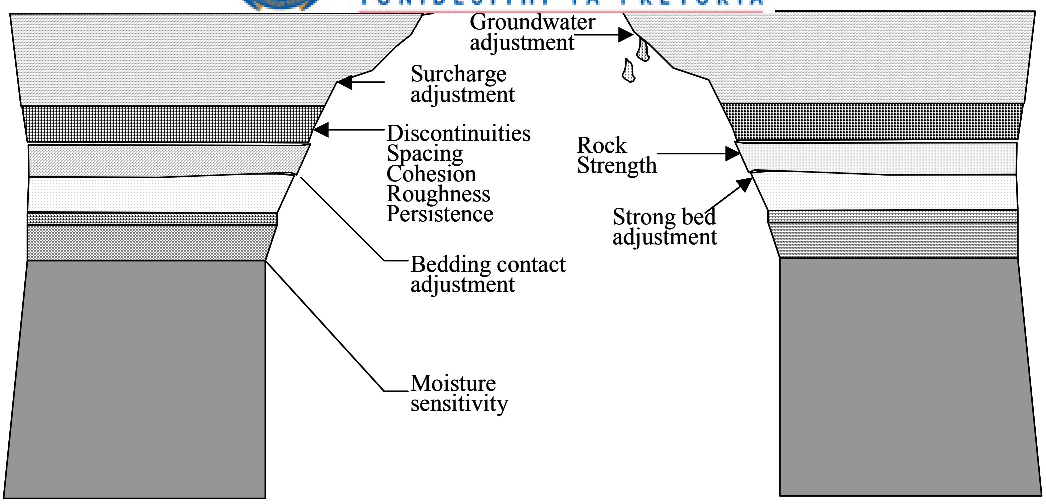


***Figure 5-1 Components of the CMRR system (after Mark and Molinda, 1994)***

The following is a summary of the factors that contribute to the final unit rating value:

a) Compressive strength of intact rock: The ball peen hammer test is used to place rock into five classes, depending on the nature of the indentation.
b) Cohesion of discontinuities: The strength of the bond between the two faces of a discontinuity is estimated by observation of roof behaviour, assisted by the chisel test.
c) Roughness of discontinuity: The surface of the discontinuity is classified as “rough”, “wavy”, or “planar” by observation.
d) Intensity of discontinuities: The average observed distance between discontinuities within a unit.
e) Persistence of discontinuity: The observed areal extent of a discontinuity plane.
f) Moisture sensitivity: Estimated from an immersion test, and only considered if significant inflows of groundwater are anticipated or if the unit is exposed to humid mine air.

After the individual unit ratings have been determined, they are summed into a single rating for the entire mine roof and adjustments are applied from the tables by taking account of the following:

- Strong beds in the bolted interval
- Number of lithologic units contacts
- Groundwater and
- Surcharge

Mark et al. (2002) modified the original CMRR described above because it could not be applied before any mining has taken place i.e. for pre-planning, as it requires underground

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observations. An entirely new system was developed to determine the CMRR from exploratory drill core using the Point Load Tests (PLT) to determine the strength parameters that account for approximately 60% of the final rating. The new system uses both diametral (parallel to bedding) and axial (perpendicular to bedding) PLT's. The diametral tests allow the estimates of bedding plane cohesion and rock anistrophy, both of which are critical to estimating susceptibility to horizontal stresses. Traditional core logging procedures are used to determine discontinuity spacing and roughness. To ensure compatibility with the original CMRR, the new rating scales were verified by comparing drill core results with nearby underground mining exposures.

A large database of strength ratings of rocks has been assembled through extensive point load testing and logging in the United States. Over 2000 PLT (both axial and diametrical) have been made on common coal measure rock types from mines representing most U.S. coal fields.

The CMRR has been determined for 97 roof exposures from 75 coal mines across the United States by Molinda and Mark (1994). All of the major U.S. coal basins are represented, with sizes ranging from small new mines to some of the largest longwall operations. The data has been partitioned to reflect the following three broad classes of roof based on a scale of 0-100: weak (0-45), moderate (45-65), and strong (65-100). Table 5-1 shows the CMRR classes with corresponding geological conditions.

***Table 5-1 CMRR classes in the U.S. (after Mark and Molinda, 1994)***

| CMRR Class | CMRR Region | Geological Conditions |
|---|---|---|
| Weak | 0-45 | Claystones, Mudrocks, Shales |
| Moderate | 45-65 | Siltstones and Sandstones |
| Strong | 65-100 | Sandstones |

CMRR has been integrated into support design programs such the "Analysis of Longwall Pillar Stability (ALPS)" program in calculation of safety factors for given coal pillar sizes based on applied loads and strength of the pillar. A similar case study in Australia by Colwell et al. (1999) has used the CMRR to develop a new methodology for chain pillar design called the "Analysis of Longwall Tailgate Serviceability (ALTS)". In both cases, statistical analysis from case histories of CMRR values have been used in conjunction with existing pillar design formulae to develop a relationship between the stability factor and roof qualities. The combination of CMRR with empirical formulae has improved the accuracy of design of gate road systems in the U.S. by integrating case histories developed through in-mine data collection techniques with numerical modelling and empirical pillar design formulae.

Mark (1999) documented the application of the CMRR to South African strata conditions since it was first introduced to the coal mining industry in 1998. Since that time, the system has been used on a limited basis owing to the fact that South African coal operations have generally been conducted in good geotechnical conditions compared to other parts of the world.

Geotechnical site investigations were conducted (van der Merwe et al. 2001) at 20 falls of ground incident sites in South African coal mines. The CMRR classification system was used to classify the roof conditions at the fall sites. In addition to that, Bieniawski's Rock Mass Rating and Laubscher's Mining Rock Mass Rating were used as comparisons with the CMRR. A stress damage survey was also undertaken to relate rock mass damage to the horizontal stress regime. In addition, a coal cleat damage was done to relate maximum horizontal stress direction to cleat orientation. All CMRR values obtain from the underground mapping sites fell in the weak class i.e. on a scale 0-100, between 0-45. Many observations from the fall of ground site mappings in South Africa were found to collate with experiences gained in the United States. However, a wide range of CMRR values were noted in some areas where roof conditions deteriorated in close proximity to major dykes or sills.

In another study by van der Merwe et al. 2001, further CMRR classification studies were carried to create a geotechnical database of the South African coal fields. The following conclusions with respect to CMRR values for South African coal mines were made:

- Roof shale's were generally within the range of 0-45 (weak)
- Sandstones were generally above the CMRR value of 45 (moderate to strong)
- Siltstones generally fell in the moderate CMRR range (45-65)

These observations correlate closely to Mark's (1994) work that siltstones and sandstones in the U.S. were moderate to strong. The CMRR has been found to be robust enough to classify and describe the roof conditions that are found in South Africa and that it was easy to learn the technique.

However, despite these advantages, in some cases the CMRR values gave a wide range in areas of high horizontal stresses and in proximity of major geological features. In one case the method over rated roof conditions (CMRR=55) in an area where orientation of major/minor geological features resulted in roof collapses due to its inability to cater for these in the unit contact adjustment van der Merwe et al. (2001).

### 5.2.1 Evaluation of CMRR

Both CMRR underground and drill core CMRR have been tested as part of this study at three South African collieries.

During this study, the greatest difficulty experienced underground with the trials of CMRR was to find nearby roof exposures with sufficient height. It was sometimes possible where there were air crossings, however, most of the time in most of the sections, CMRR could not be applied. Therefore, the underground visits suggested that for quick and comparative results, a detailed rating system that requires data on roof stratification can only be used in the planning stage on borehole cores.

One other problem experienced underground was the effect of a single discontinuity which could cause significant damage to the roof. Because CMRR only took sets of discontinuities into account, it was observed that the effect of single joint together with the direction of it should be included in a coal mine roof rating system. Van der Merwe et al. (2001) showed that 37 per cent of 182 falls of ground in South African collieries, which were investigated during the course of the project, were caused by mainly single joints. It is also found that the blasting damage in the roof should be included in a coal mine roof rating system. In addition, van der Merwe et al. (2001) also highlighted that less than 10 per cent of 182 falls of ground in South African collieries were caused by high horizontal stress. Although it is not a major cause of falls of ground, an adjustment factor in a rating system to account for high horizontal stress is required.

Other important shortcomings of CMRR were the rated height into the roof and the stratigraphic position of weak layers. The first 2.0 m into the roof is usually rated in South Africa collieries. One advantage of this is that, if the rating system is used for comparison purposes, it is important to compare the same height in each rating. Also, the effect of soft layers high into the roof, even if significantly thinner than those lower in the sequence, can affect the stability of roof.

During this evaluation study, there was difficulty in comparing underground CMRR with locally developed systems owing to the fact that in the collieries visited, their systems were not developed for rating the roof, but for planning purposes. Therefore, a direct comparison between the locally used colliery-based systems and CMRR could not be carried out. However, impact splitting testing and CMRR were compared on surface using drill cores. This highlighted the shortcoming of CMRR with respect to the relative positions of stiff and soft layers in the roof. Figure 5-2 shows three different 0.9 m long cores. Each core contains three different 0.3 m long layers, namely, sandstone, shale and siltstone, but set up in different sequences, e.g. sandstone is positioned at the top, middle or bottom of the different core runs respectively.

The results obtained from the CMRR were exactly the same for all three cores. This indicated that the CMRR does not consider the position of soft or stiff layers within the roof strata. However, impact splitting tests resulted in three different ratings based on the position of stiff sandstone layer into the roof that affects the stability of the roof. This indicates that the CMRR rates the quality of roof as a whole without considering the positions of different layers in the roof, and hence the likelihood and potential severity of the instability. This has major implications in South African collieries, since in many cases the support design is based on the stiffness of the immediate roof layer. Lastly, CMRR requires skilled personnel and some degree of training.

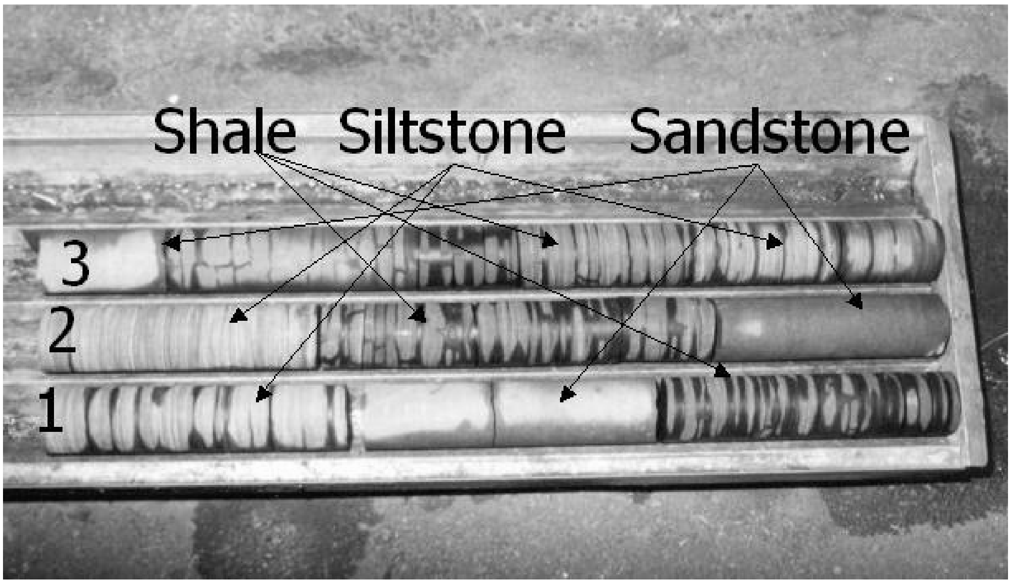


***Figure 5-2 Cores used for CMRR and impact splitting testing***

In summary, the shortcomings of CMRR, which were identified during the evaluation study of CMRR, are summarized below:

- Exposure into the roof is required (underground CMRR only)
- Only the bolted height is rated. In South Africa, 2.0 m into the roof is the height that is usually rated. Typical bolted heights in South Africa are less than 2.0 m.
- Although sets of joints have been considered in CMRR, single joints should also be included.
- Joint orientation is not taken into account (underground CMRR only).
- Stress adjustment is required in the rating system to account for the influence of high horizontal stress (underground CMRR only)

- No adjustment is made for the effects of blasting (underground CMRR only)
- The position of soft or hard layers into the roof is not taken into account (both underground and borehole core CMRR)
- Skilled personnel are required to carry out ratings (both underground and borehole core CMRR)
- Subjectivity in the rating is not entirely eliminated

## 5.3 Rating systems being used in South African collieries

### 5.3.1 Rating systems developed for planning purposes

Van der Merwe (2001) developed the first roof rating system in South Africa in 1980, using Rock Quality Designation (RQD). In this rating system the critical height into the roof was taken as 2.0 m. This height of the roof was initially rated with RQD. Following a splitting test conducted with a chisel at regular distances along the core, RQD was re-applied and final results were compared with the initial results. The final rating was then obtained based on the difference between the initial and final RQDs. Due to possible discrepancies resulting from the use of chisels with different geometries and forces, van der Merwe (1989) developed a standard chisel for all roof rating tests. A summary of the rating systems that have been documented for use in coal mining in South African is given in Table 5-2.

Jermy and Ward, 1988 conducted an investigation into relating geotechnical properties of various sedimentary facies to their observed underground behaviour to quantify geological factors that affect roof stability in coal mines. Twenty-four distinct facies types were determined from borehole cores from a number of collieries throughout South Africa. A database of 10 000 tests from core samples was compiled from the Waterberg, Witbank, Highveld, Eastern Transvaal, Klip River, Utrecht and Vryheid Coalfields. The results from the tests have shown that those facies with lower direct tensile strengths generally gave rise to unstable roof conditions. Furthermore, the low direct tensile strengths of the argillaceous facies were found to be very important when considering the behaviour of these rocks underground. The arenaceous facies were found to have higher average direct tensile strengths. However, the authors found that this can be reduced dramatically by the presence of argillaceous or carbonaceous partings within the rock which can affect the roof stability. Other tests that were included in the assessment were the Brazilian Strength Tests (BTS) and the Uniaxial Compressive Strength (UCS). Descriptions of sedimentary facies and a summary of their underground properties are given in Table 5-3.

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***Table 5-2 A summary of some classification systems used in South African coal mining and their main applications***

| Name of classification system | Form and Type** | Main Applications | Reference |
|---|---|---|---|
| Roof and floor classification for collieries | Descriptive form | For quantification of geological factors that affect roof stability | Jermy and Ward, 1988 |
| Duncan Swell and Slake Durability tests | Numerical and behaviouristic form Functional type | Quantification of floor conditions | Buddery and Oldroyd, 1992 |
| Impact splitting test | Descriptive and behaviouristic form Functional type | Coal roof characterization and support design | Buddery and Oldroyd, 1992 |
| CMRR | Descriptive and behaviouristic form Functional type | Coal roof characterization and support design. | Molinda and Mark, 1994 |
| Section physical risk and performance rating | Descriptive Functional type | Classification of adherence to mine standards and physical rating | Oldroyd and Latilla, 1999 |

**Definition of the Form and Type:
*Descriptive form*: the input to the system is mainly based on descriptions
*Numerical form*: the input parameters are given numerical ratings according to their character
*Behaviouristic form*: the input is based on the behaviour of the rock mass.
*General type:* the system is worked out to serve as a general characterization
*Functional type:* the system is structured for a special application (for example for rock support recommendation)

***Table 5-3 Description of sedimentary facies and summary of their underground properties***

| FACIES | DESCRIPTION | PROPERTIES OF ROCK STRATA UNDERGROUND |
|---|---|---|
| 1 | Massive dark grey to black carbonaceous siltstone. | Very poor roof and floor strata due to low tensile strength and deteriorates rapidly upon exposure. Roof falls common and floor heave occurs when depth of mining exceeds 150 m. |
| 2 | Lenticular-bedded siltstone with discontinuous ripple cross lamination. Resembles lenticular bedding of Reineck and Wunderlich (1986). | |
| 3 | Alteration of 1 cm thick layers of flat laminated siltstone and fine grained sandstone. | |
| 4 | Flaser bedded siltstone and fine grained sandstone as described by Reineck and Wunderlich (1968). | Reasonable roof strata which deteriorates upon exposure giving rise to spalling from the roof. |
| 5 | Ripple cross laminated fine-grained grey feldspathic sandstone. | Reasonable roof strata, although localised roof falls do occur due to parting along silt drapes. Durability good. |
| 6 | Ripple cross laminated fine-grained grey feldspathic sandstone with silt drapes and grit bands. | |
| 7 | Massive fine grained greyish white feldspathic sandstone. | Very competent floor and roof strata due to low porosity and high tensile strength. |
| 8 | Fine grained greyish white feldspathic sandstone with planar/trough crossbeds. | |
| 9 | Massive medium grained white feldspathic sandstone. | Good roof and floor strata with fairly high tensile strengths. Sometimes creates problems due to poor goafing ability in stooping areas. |
| 10 | Medium grained white feldspathic sandstone with planar/trough crossbeds | |
| 11 | Massive coarse grained white feldspathic sandstone. | Good roof and floor strata. Decomposes under prolonged saturation giving rise to stability problems. |
| 12 | Coarse grained white feldspathic sandstone with planar/trough crossbeds. | |
| 13 | Intensely bioturbated carbonaceous siltstone or fine-grained sandstone. | Deteriorates rapidly upon exposure and saturation to give roof and floor instability. |
| 14 | Medium to coarse-grained feldspathic sandstone with irregular carbonaceous drapes and slump structures. | No information available. |
| 15 | Highly carbonaceous silty sandstone. | No information available. |
| 16 | Whitish brown calcrete. | Not applicable. |
| 17 | Highly weathered creamy orange to grey Beaufort (?) mudstone. | |
| 18 | Unweathered grey Beaufort (?) mudstone. | |
| 19 | Massive khaki to grey mudstone associated with diamictite. | |
| 20 | Dark greyish black gritty diamictite with angular 0-4 mm matrix supported clasts | |
| 21 | Dark greyish black pebbly diamictite with , angular matrix supported clasts > 4 mm diameter. | |
| 22 | Coal mixed dull and bright. | More stable roof rock than facies 1-3. |
| 23 | Mixed coal and mudstone. | Not applicable. |
| 24 | Massive greyish black carbonaceous mudstone associated with coal seam middling. | |

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Buddery and Oldroyd (1992) developed a roof and floor classification system for collieries. The following philosophy was applied in devising a suitable classification system:

- The rock property tests should be related to the expected mode of failure of the strata.
- The whole spectrum of strata should be tested with particular emphasis being placed on obtaining the properties of the weakest material.
- Large numbers of tests should be able to be conducted simply, quickly, at low cost and in-house.

Roof failure in South African coal mines is strongly related to the frequency of laminations or bedding planes. In their roof classification, Buddery and Oldroyd (1992) considered a Coal Rock Structure Rating (CRSR) system to classify the roof condition. Tests to indicate the propensity of the laminations or bedding planes to open and separate will therefore be ideal for planning. The tests should indicate the mode of failure of the roof and it should be easy for a large number of the tests to be conducted. This was initially based on three parameters: RQD, the results of impact splitting tests, and a parameter related to joint condition and groundwater. Due to the impracticality of satisfactorily distinguishing between drilling-induced and natural fractures in the coal measures strata, the RQD parameter was discarded from the system. The third parameter proved to be difficult to determine irrespective of the roof type. It was, therefore, decided to confine the determination of roof ratings to the results of impact splitting tests.

The impact splitting test involves imparting the same impact to the core at 20 mm intervals. The resulting fracture frequency is then used to determine a roof rating. The instrument shown in Figure 5-3 consists of an angle iron base which holds the core. Mounted on this is a tube containing a chisel with a mass of 1.5 kg and a blade width of 25 mm. The chisel is dropped onto the core from a constant height according to core size, 100 mm for a 60 mm diameter core and 64 mm for 48 mm diameter core. The impact splitter caused weak or poorly cemented bedding planes and laminations to open, thus giving an indication of the likely *in situ* behaviour when subjected to bending stresses.

***Figure 5-3 The Impact splitting equipment***

It is suggested that, when designing coal mine roof support, 2.0 m of strata above the immediate roof should be tested. If the roof horizon is in doubt, then all strata from the lowest likely roof horizon to 2.0 m above the highest likely roof horizon are tested so that all the potential horizons may be compared. In this classification system, the strata are divided into geotechnical units. The units are then tested and the mean fracture spacing for each unit is obtained. An individual rating for each unit is determined by using one of the following equations:

$$\begin{aligned} \text{For } fs &\leq 5 \quad \text{rating} = 4fs \\ \text{For } fs &> 5 \quad \text{rating} = 2fs+10 \end{aligned} \qquad [5\text{-}1]$$

Where *fs* = fracture spacing is in *cm*

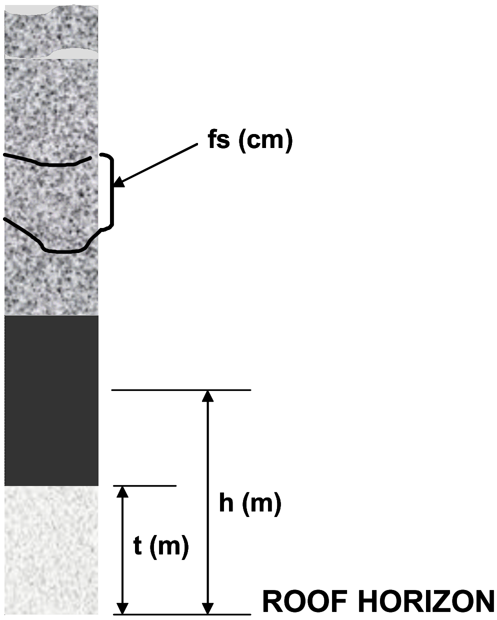


***Figure 5-4 Impact splitting unit rating calculation***

This value is then used to classify the individual strata units into rock quality categories as shown in Table 5-4. For coal mine roofs, the individual ratings are adjusted to obtain a roof rating for the first 2.0 m of roof. It was stated that the immediate roof unit will have a much greater influence on the roof stability and consequently the unit ratings are weighted according to their position in the roof by using the following equation:

$$Weighted\ rating\ =\ rating\ x\ 2(2\text{-}h)\ t \qquad [5\text{-}2]$$

Where $h$ is mean unit height above the roof in metres and $t$ is thickness of unit in metres (Figure 5-4).

The weighted ratings for all units are then totalled to give a final roof rating. Buddery and Oldroyd (1992) concluded that good agreement between expected and actual roof conditions has been found when using this rating system.

Latilla et al. (2002) revised the unit and coal roof classification system, and recommended the following Table for classification of coal mine roofs:

***Table 5-4 Unit and coal roof classification system (after Latilla et al, 2002)***

| Unit Rating | Rock Class | Roof Rating |
|---|---|---|
| < 9 | Very poor | < 34 |
| 10 – 13 | Poor | 35 – 51 |
| 14 – 19 | Moderate | 52 – 75 |
| 20 – 28 | Good | 76 – 113 |
| 29 – 42 | Very good | 114 – 167 |
| > 42 | Excellent | > 167 |

In addition, Latilla et al. (2002) suggested an adjustment factor to take into account areas where the immediate roof is coal. The unit rating is multiplied by 1.56, which is the density of sandstone (2500 kg/m3) divided by coal density (1600 kg/m3).

Based on this rating system the support patterns listed in Table 5-5 are adopted together with a special “current-with-mining assessment” technique to adapt to changing roof conditions, Latilla et al. (2002).

Sasol Coal also developed a roof rating system based on fall of ground accidents. Analyses of fall of ground (FOG) accidents in group collieries indicated that almost all such accidents occurred near dykes and underneath rivers. The collieries have been divided into three groups indicating the roof conditions based on these two criteria. These areas are marked on mine plans as Class 'C', Class 'B' and Class 'A'. The worst and the best ground conditions are expected in Class 'A' and Class 'C' respectively. In Class 'C' areas, a spare roofbolter and tell-tales should be available to cater for and identify possible roof deterioration.

***Table 5-5 Estimated support requirements for different roof classifications (after van Wijk, 2004)***

| Roof condition | Bord width (m) | Typical systematic support | | | |
|---|---|---|---|---|---|
| | | **Type** | **Length (m)** | **Pattern** | **Distance between rows of bolts (m)** |
| **Excellent** | 7 | M16 point anchor | 0.9 or 1.2 | Spot bolting false roof | N/A |
| **Very Good** | 6.5 to 7 | M16 point anchor | 1.2 | Spot bolting and 5 bolts per intersection only | N/A |
| **Good** | 6 to 6.5 | M16 point anchor | 1.2 or 1.5 | 5 bolts per intersection and 2 per row in bords | 2 to 2.5 |
| **Moderate** | 5.5 to 6 | M16 or M20 full column resin | 1.5 or 1.8 | 9 bolts per intersection and 3 per row in bords | 1.5 to 2 |
| **Poor** | 5 to 5.5 | M20 full column resin | 1.8 | 16 bolts per intersection and 4 per row in bords. Steel straps may be necessary | 1 to 1.5 |
| **Very Poor** | <5 | Specialised support, e.g. 1.8m M20 full column resin bolts and/or cable anchors with steel straps. Cable trusses, cluster stick packs or shotcrete may also be required | ≥1.8 | As dictated by conditions. Typically 5 bolts per row with steel straps. Often 9 cables in intersections. | <1 |

On each special area plan, a borehole log is also attached to indicate to mining personnel the roof conditions in the area. This also assists mining personnel in determining what length of roof bolt to use in the area. The same mining group has also developed a rating system to be used on borehole cores in greenfield areas, called Percentage Lamination Plan. This plan assists mining personnel in determining;

- the thickness of laminated material,
- whether the laminated stratum is high or low in the roof,
- whether the lamination is such that intersection failure can occur,

- whether the section is approaching ground where drastic changes in roof conditions can occur.

This plan indicates the percentage laminated strata in the direct roof and is available in the following ranges: the first metre of roof, the second metre of roof and the first two metres of roof.

There are also rating systems used in South Africa that are empirical correlations between particular features and roof behaviour based on the local geology. These systems are usually based on experience of mining personnel or especially geologists. If a specific layer or the position of a layer caused problems underground, these layers and/or position of these layers usually formed the rating systems. Experienced geologist identifies the significant layer and its position in the roof, during the logging of boreholes. This information is then marked on mine plans and its position referred to as Roof Hazard Plans. In geology based rating systems, the thickness of particular layers is also found to be important. Therefore, for some mines, the roof rating is based on the thickness of particular layers, such as sandstone, shale or siltstone, and the roof support pattern is determined by the assessed quality of the roof. It was also found that geological discontinuities are important and play a major role in the quality of roof, therefore, some mines adapted rating systems based on these features.

As the mines had problems with a certain rock type or with the thickness of certain rock type, they extended their rating systems by including them in their systems. Because these systems were purely based on years of experience, an appropriate universal system should correlate well with these experienced-based systems.

A review of the rating systems being used in South Africa highlighted that roof rating systems are being used mainly for planning purposes, and not to determine the changing conditions underground. However, rating systems have also been developed in South Africa by Ingwe Coal (Oldroyd and Latilla, 1999), in which support systems are changed based on on-going evaluation of changing underground conditions.

### 5.3.2 Proactive rating systems developed to identify changing conditions

Mechanised mining allows sections to be developed at a rapid rate, typically more than 1000 m per month for most sections, this can result in a variety of conditions being encountered in a single section in a very short time. Van Wijk et al. (2002) identified a number of accidents in Ingwe Coal (a division of BHP Billiton Energy Coal) mines that were caused primarily by the inability to recognise changing conditions and therefore failing to apply necessary counter measures timeously.

In order to identify the changing ground conditions, van Wijk et al. (2002) documented two underground rating systems: the "Section Physical Risk Rating" for measuring the physical conditions and the "Section Performance Rating" for determining how well the underground section personnel response to these conditions. Both forms are essentially risk matrices defining various scenarios, each with a certain weighting.

The section physical risk rating form is a basic questionnaire requesting information regarding geological conditions relevant to roof and sidewall stability, the mining method, and the support system, together with other geological information to determine a physical ranking that ensures the total system is examined. The section performance rating form is designed to measure how conditions determined by the section physical risk rating are being addressed. Furthermore, the form also measures compliance with the support rules and strata control standards. Both forms can be easily adapted for specific conditions. Should geological discontinuities, for example, represent a major problem in a particular area or for a specific mining method, then the importance of these features may be highlighted as a separate item with its own sub-divisions or by changing the weighting.

In summary, the following are some of the benefits of using the section physical and performance risk ratings:

- The rating forms enable quantification of previously subjective observations.
- Different auditors, i.e. shift supervisors, mine overseers and rock engineers, use the same format. This allows meaningful comparisons to be made in individual sections.
- A visit (audit) is structured such that people observe and record all potential hazards. It enables trends to be monitored and forms an integral part of the section management plan.

Van Wijk et al. (2002) describes the impact splitting tests, section performance rating and physical risk ratings as a system that can be used during the planning stage and assigning appropriate support patterns; for identifying changing conditions while mining; determining the best reaction to those conditions.

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### 5.3.3 Colliery specific systems being used in South Africa

A number of hazard rating systems are used by the coal mines in South Africa. Some of these have already been documented but in most cases the systems are designed and implemented by the individual mines themselves. In light of this, it was necessary to investigate these different hazard systems by conducting visits to the coal mines. It was decided that this task would be approached in three stages:

1. Documenting the colliery's hazard rating system;
2. Applying an existing system to test it against the colliery's system.
3. Comparison of results of the existing systems to the colliery's rating system.

One of the initial tasks for this study was to devise an effective method to directly compare the different rating systems used in different collieries. The reason for this is that most of the systems are not documented and as already mentioned and differ from one mine to another. It is for this reason that impact splitting was considered as the most effective system to apply at each mine in order to test it against the mine's system and also to test one mine's results against another mine. The section performance rating and physical risk ratings were also conducted underground to test their applicability at each colliery.

The research was conducted at eight collieries in the Witbank and Highveld Coalfields. This section of this thesis presents the results of the investigations at each colliery.

## 5.4 Geotechnical testing at different collieries

As mentioned above, a number of rating systems are used by the coal mines in South Africa. These systems are usually based on experience of local mining personnel, and implemented by the individual mines themselves. A series of impact splitting tests were therefore conducted and compared against the mines individual systems in order to determine the reliability and repeatability of impact splitting tests against the systems that are developed over many years of experienced on the mines. Tests were conducted at six different mines.

The following lithological codes are used in the following tables:

| | |
|---|---|
| C | : Coal |
| F | : Shale |
| S | : Sandstone |
| S/f | : Sandstone with shale bands |

| | |
|---|---|
| S/F | : Sandstone/Shale interlaminated |
| S/s | : Sandstone/Siltstone (Predominantly Sandstone) |
| SC | : Sandstone/Coal (Predominantly Sandstone) |
| SF | : Shaley sandstone/siltstone |

### 5.4.1 Colliery 'A'

At this colliery, a rating system implemented by the geology department is used to predict the anticipated underground conditions for planning. The plan used for support design is based on the thickness of the gritstone (coarse grained sandstone), which is a strong stratum that can act as a self-supporting beam and is referred to as the Roof Grit Plan. The grit plan was divided into four-thickness categories and classified. Support recommendations are then made as shown in Table 5-6. The underlying principle in terms of support recommendations is that the thinner the grit, the longer should be the anchorage length. The density of support is also increased as the grit thickness reduces. The geology department also makes use of a Point Load Tester to measure the strength of the rock types in the roof and the floor. This information is then used mainly for contamination and floor cutability purposes more than classification of the grit strength.

***Table 5-6 Roof grit hazard classification used at Colliery 'A'***

| Roof Grit | Classification | Typical Support |
|---|---|---|
| No Grit | Very Poor | W-straps with cable anchors |
| < 0.5 m Grit | Poor | 1.8 m Full Column Resin, with W-straps for Slips |
| 0.5 m - 1.0 m Grit | Moderate | 1.2 m – 1.5 m Full Column Resin |
| 1.0 m to 2.0 m Grit | Good | 1.2 m Full Column Resin |
| > 2.0 m Grit | Very Good | 0.9 m Full Column Resin |

The roof grit plan is demarcated in different colours representing different roof grit thicknesses and the information is superimposed on the underground mining plan. At each section, a separate underground section plan is provided that incorporates the anticipated roof conditions from the Roof Grit Plan, as well as geological structures, mining parameters, methane contents and horizontal stress mapping. The underground section plan is approved by the mine surveyor, mine geologist, assistant manager, planning officer and environmental officer to ensure that all parameters are correctly represented on the plan.

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A comparative study was conducted on three borehole drill cores, about 100 m from each other on the No 2 Seam.

Table 5-7 to Table 5-9 show the results of impact splitting of the three borehole drill cores. The mine geologists classified borehole drill core ARN 4968 as Roof Grit of 2.19, i.e. “Good” roof. From, Table 5-7 the final rating of 232 from impact splitting classifies the borehole drill core as “Excellent” roof.

Figure 5-5 shows a unit from the roof before impact splitting. The initial fractures are counted before the impact splitting, i.e. one on this case. Figure 5-6 shows the same unit after impact splitting with 3 final fractures.

***Figure 5-5 A fine to medium grained sandstone or “grit” unit before impact splitting, taken from borehole ARN 4968***

***Figure 5-6 A fine to medium grained sandstone or “grit” unit after impact splitting, taken from borehole ARN 4968***

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***Table 5-7 Impact splitting results at Colliery 'A', No 2 Seam, borehole ARN 4968***

| Depth (m) | Thickness (cm) | Lithology | Initial Fractures | Final Fractures | Fracture Spacing (cm) | Unit Rating | Weighted Rating | Remarks |
|---|---|---|---|---|---|---|---|---|
| 46.5 | 13.2 | S/F | 1 | 6 | 2.2 | 8.8 | 0.1 | Very Poor |
| 46.7 | 20 | S/F | 1 | 2 | 10.0 | 30.0 | 2.4 | Good |
| 47 | 25.2 | S | 1 | 4 | 6.3 | 22.6 | 5.4 | Moderate |
| 47.3 | 34.5 | S | 1 | 2 | 17.3 | 44.5 | 22.3 | Very Good |
| 47.6 | 24.5 | S | 1 | 1 | 24.5 | 59.0 | 31.2 | Very Good |
| 47.8 | 24 | S | 1 | 2 | 12.0 | 34.0 | 20.9 | Very Good |
| 48.4 | 61.6 | S | 1 | 2 | 30.8 | 71.6 | 149.3 | Very Good |
| | | | | | | | **Final Rating** | |
| | | | | | | | **232** | **Excellent** |

Table 5-8 and Table 5-9 show the results from impact splitting of the other two borehole drill cores. The final ratings of borehole drill cores ARN 4974 and ARN 4975 are 274 - "Excellent" roof - and 199 - "Excellent" roof. The mine geologists classified the borehole drill cores as Roof Grit of 1.95 – 2.09 "Good" roof. These results show a good correlation between impact splitting tests and the roof grit plan classification. The advantage of impact splitting is that it quantifies the roof condition as opposed to the mere description of the thickness of the gritstone. Moreover, where grit layer is not so obvious, the mine's system may result in errors due to the subjectivity of the assessment technique.

***Table 5-8 Impact splitting results at Colliery 'A', No 2 Seam, borehole ARN 4974***

| Depth (m) | Thickness (cm) | Lithology | Initial Fractures | Final Fractures | Fracture Spacing (cm) | Unit Rating | Weighted Rating | Remarks |
|---|---|---|---|---|---|---|---|---|
| 43.6 | 32.5 | S/F | 1 | 5 | 6.5 | 23.0 | 3.6 | Moderate |
| 43.9 | 36 | S/F | 1 | 5 | 7.2 | 24.4 | 9.1 | Moderate |
| 44.3 | 41.8 | S | 1 | 2 | 20.9 | 51.8 | 38.6 | Very Good |
| 44.8 | 45.3 | S | 1 | 2 | 22.7 | 55.3 | 68.8 | Very Good |
| 45.2 | 44 | S | 1 | 1 | 44.0 | 98.0 | 153.5 | Very Good |
| | | | | | | | **Final Rating** | |
| | | | | | | | **274** | **Excellent** |

Underground visits were also conducted to assess adherence to the underground anticipated physical conditions and mine standards using physical rating system and performance rating system. These systems were successful in identifying possible hazards but because they originated from a different mine, some parameters could not be recorded owing to different

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specifications e.g. Colliery 'A' standards of support spacing are not included in the rating systems.

***Table 5-9 Impact splitting results at Colliery 'A', No 2 Seam, borehole ARN 4975***

| Depth (m) | Thickness (cm) | Lithology | Initial Fractures | Final Fractures | Fracture Spacing (cm) | Unit Rating | Weighted Rating | Remarks |
|---|---|---|---|---|---|---|---|---|
| 44.86 | 15.5 | F/S | 1 | 4 | 3.9 | 15.5 | 0.4 | Very Poor |
| 44.96 | 10 | F/S | 1 | 3 | 3.3 | 13.3 | 0.5 | Very Poor |
| 45.20 | 24.2 | F/S | 1 | 6 | 4.0 | 16.1 | 2.9 | Very Poor |
| 45.31 | 11.5 | F/S | 1 | 3 | 3.8 | 15.3 | 2.0 | Very Poor |
| 45.89 | 57.5 | S | 1 | 4 | 14.4 | 38.8 | 40.1 | Poor |
| 46.10 | 21 | S | 1 | 2 | 10.5 | 31.0 | 16.8 | Poor |
| 46.33 | 23.2 | S | 1 | 2 | 11.6 | 33.2 | 23.3 | Poor |
| 46.70 | 37 | S | 1 | 1 | 37.0 | 84.0 | 112.8 | Very Good |
| | | | | | | | **Final Rating** | |
| | | | | | | | **199** | **Excellent** |

### 5.4.2 Colliery 'B'

At Colliery 'B', a roof hazard plan only exists for the No 5 Seam. The hazard plan is based mainly on geological structures, roof type above the coal seam (from boreholes), horizontal stresses, and surface structures e.g. pans. Geological structures include dykes and sills with associated burnt coal areas. The roof type above the coal seam is described from exploration boreholes and is classified from the lithological description of the borehole as shown in Table 5-10.

Horizontal resistivity measurements are carried out on surface to determine the depth of weathering to assist in mine planning, considering that weathering allows increased water content which can affect the strength of the roof. Individual boreholes were analysed and the classification of normal, poor and bad roof is identified according to the composition of the immediate roof and the overlying strata.

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***Table 5-10 Roof hazard classification at Colliery 'B'***

| **Classification** | **Roof type** |
|---|---|
| Normal roof | Shale or siltstone of more than 30 cm thick overlain by sandstone |
| Poor | Interlaminated, laminated, fissile and micaceous sandstone, siltstone and shale less than 30 cm |
| Bad roof | Dolerite intrusions, deep weathering of the roof and faults |

The hazards identified in the roof hazard plan are included in all section plans issued by the survey department. When mining towards an area that has been demarcated in the roof hazard plan, various procedures come into effect in terms of personnel awareness and roof support.

A comparative study was conducted on a total of five borehole drill cores, three from the No 5 Seam and two from the No 2 Seam. These borehole cores were mainly drilled for future planning and thus their numbers are the temporal numbers used by the drillers. The results of impact splitting of the five borehole drill cores from No 5 Seam and No 2 Seam are presented from Table 5-11 to Table 5-15.

Figure 5-7 shows an example of the borehole drill core of the Sandstone/Shale interlaminated roof from the No 5 Seam. In Figure 5-8 the weaker roof composed mainly of shale is shown.

***Figure 5-7 Borehole drill core from Colliery 'B', No 5 Seam***

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***Figure 5-8 Borehole drill core from Colliery 'B', No 2 Seam***

***Table 5-11 Impact splitting results at Colliery 'B', No 5 Seam, borehole H45S5***

| **Depth** (m) | **Thickness** (cm) | **Lithology** | **Initial Fractures** | **Final Fractures** | **Fracture Spacing** (cm) | **Unit Rating** | **Weighted Rating** | **Remarks** |
|---|---|---|---|---|---|---|---|---|
| 38 | 14.5 | S | 1 | 2 | 7.3 | 24.5 | 5.2 | Moderate |
| 38.1 | 12.7 | S/F | 1 | 5 | 2.5 | 10.2 | 2.2 | Poor |
| 38.3 | 14.5 | S/F | 1 | 5 | 2.9 | 11.6 | 3.5 | Poor |
| 38.4 | 16.4 | S/F | 1 | 4 | 4.1 | 16.4 | 6.0 | Poor |
| 38.6 | 12.5 | S | 1 | 1 | 12.5 | 35.0 | 11.7 | Very Good |
| 38.7 | 14.2 | S | 1 | 1 | 14.2 | 38.4 | 15.6 | Very Good |
| 38.8 | 12 | S/F | 1 | 2 | 6.0 | 22.0 | 8.1 | Moderate |
| 39 | 14.5 | S/F | 1 | 2 | 7.3 | 24.5 | 12.3 | Moderate |
| 39.1 | 11.5 | S/F | 1 | 2 | 5.8 | 21.5 | 9.1 | Moderate |
| 39.2 | 11 | S | 1 | 1 | 11.0 | 32.0 | 13.7 | Very Good |
| | | | | | | | **Final** | **Good** |
| | | | | | | | **87** | **Moderate** |

The final rating from impact splitting of borehole drill core H45S5 is 87, which is classified as "Good" roof. A similar classification of "Good" was obtained from the final ratings of drill cores H49S5 and H50S5 i.e. 87 and 84. These results from the three borehole drill cores could not be directly compared to the colliery's rating system due to the fact that the borehole drill cores were done for future planning purposes by a drilling contractor. Furthermore, due to staff changes during the course of this study at the mine, the new geologist had difficulty in learning their rating system. However, impact splitting results show a good correlation between each of the three tests, which were taken in maximum possible proximity i.e. were spaced at less than 500 m.

***Table 5-12 Impact splitting results at Colliery 'B', No 5 Seam, borehole H49S5***

| Depth (m) | Thickness (cm) | Lithology | Initial Fractures | Final Fractures | Fracture Spacing (cm) | Unit Rating | Weighted Rating | Remarks |
|---|---|---|---|---|---|---|---|---|
| 37.9 | 14.5 | S | 1 | 1 | 14.5 | 39.0 | 3.7 | Very Good |
| 38.1 | 17 | S/F | 1 | 6 | 2.8 | 11.3 | 2.0 | Poor |
| 38.2 | 10.5 | S/F | 1 | 3 | 3.5 | 14.0 | 1.9 | Poor |
| 38.3 | 12 | S/F | 1 | 3 | 4.0 | 16.0 | 2.8 | Poor |
| 38.5 | 17.1 | S/F | 1 | 4 | 4.3 | 17.1 | 5.3 | Moderate |
| 38.6 | 11.5 | S/F | 1 | 4 | 2.9 | 11.5 | 2.8 | Poor |
| 38.8 | 18 | S/F | 1 | 4 | 4.5 | 18.0 | 7.8 | Moderate |
| 39 | 22 | S | 1 | 4 | 5.5 | 21.0 | 12.8 | Moderate |
| 39.4 | 35.2 | S | 1 | 4 | 8.8 | 27.6 | 33.5 | Good |
| 39.5 | 15 | S | 1 | 3 | 5.0 | 20.0 | 11.6 | Moderate |
| | | | | | | | **Final Rating** | |
| | | | | | | | **84** | **good** |

From Table 5-14 and Table 5-15, the final ratings obtained from the No 2 Seam are 21 and 15 which indicate "Very Poor" roof in each case. The weakness of the shale in this case made it difficult to rate up to 2 m into the roof due to the shale being easily broken by merely picking it up from the borehole drill core box. However, the results show the advantage of impact splitting over the colliery's system in its ability to readily quantify the roof instead of a mere description that can change from one persons perception to another.

***Table 5-13 Impact splitting results at Colliery 'B', No 5 Seam, borehole H50S5***

| Depth (m) | Thickness (cm) | Lithology | Initial Fractures | Final Fractures | Fracture Spacing (cm) | Unit Rating | Weighted Rating | Remarks |
|---|---|---|---|---|---|---|---|---|
| 37.6 | 15 | S | 1 | 2 | 7.5 | 25.0 | 3.9 | Moderate |
| 37.8 | 14.7 | S/F | 1 | 7 | 2.1 | 8.4 | 1.8 | Very Poor |
| 37.9 | 16 | S/F | 1 | 5 | 3.2 | 12.8 | 3.4 | Poor |
| 38.1 | 12.6 | S/F | 1 | 5 | 2.5 | 10.1 | 2.6 | Poor |
| 38.2 | 10.5 | S/F | 1 | 2 | 5.3 | 20.5 | 4.9 | Moderate |
| 38.3 | 12.4 | S | 1 | 1 | 12.4 | 34.8 | 10.7 | Very Good |
| 38.5 | 20.8 | S/F | 1 | 2 | 10.4 | 30.8 | 17.9 | Good |
| 38.7 | 19.5 | S/F | 1 | 4 | 4.9 | 19.5 | 12.2 | Moderate |
| 38.8 | 16.4 | S/F | 1 | 2 | 8.2 | 26.4 | 14.9 | Moderate |
| 39 | 15.4 | S/F | 1 | 2 | 7.7 | 25.4 | 15.0 | Moderate |
| | | | | | | | **Final Rating** | |
| | | | | | | | **87** | **Good** |

***Table 5-14 Impact splitting results at Colliery 'B', No 2 Seam, borehole P4S2***

| Depth (m) | Thickness (cm) | Lithology | Initial Fractures | Final Fractures | Fracture Spacing (cm) | Unit Rating | Weighted Rating | Remarks |
|---|---|---|---|---|---|---|---|---|
| 59.3 | 12.2 | F | 1 | 6 | 2.0 | 8.1 | 2.5 | Very Poor |
| 59.4 | 10.1 | F | 1 | 5 | 2.0 | 8.1 | 2.2 | Very Poor |
| 59.5 | 14.5 | F | 1 | 8 | 1.8 | 7.3 | 3.0 | Very Poor |
| 59.6 | 12.3 | F | 1 | 5 | 2.5 | 9.8 | 3.7 | Very Poor |
| 59.8 | 11.5 | F | 1 | 6 | 1.9 | 7.7 | 3.1 | Very Poor |
| 59.9 | 11.2 | F | 1 | 6 | 1.9 | 7.5 | 3.1 | Very Poor |
| 60 | 13 | F | 1 | 7 | 1.9 | 7.4 | 3.7 | Very Poor |
| | | | | | | | **Final Rating** | |
| | | | | | | | **21** | **Very Poor** |

***Table 5-15 Impact splitting results at Colliery 'B', No 2 Seam, borehole P3S2***

| Depth (m) | Thickness (cm) | Lithology | Initial Fractures | Final Fractures | Fracture Spacing (cm) | Unit Rating | Weighted Rating | Remarks |
|---|---|---|---|---|---|---|---|---|
| 54.6 | 12.5 | F | 1 | 6 | 2.1 | 8.3 | 3.2 | Very Poor |
| 54.8 | 13 | F | 1 | 6 | 2.2 | 8.7 | 3.9 | Very Poor |
| 54.9 | 10.5 | F | 1 | 4 | 2.6 | 10.5 | 4.1 | Poor |
| 55.0 | 12.5 | F | 1 | 6 | 2.1 | 8.3 | 4.0 | Very Poor |
| | | | | | | | **Final Rating** | |
| | | | | | | | **15** | **Very Poor** |

### 5.4.3 Colliery 'T'

At Colliery 'T', hazard plans are based on analyses of fall of ground accidents. The sections have been divided into three groups depending on the position of the section relative to dykes and surface rivers. These areas are marked on mine plans as Class 'C', Class 'B' and Class 'A'. The guidelines for maximum bord width and cut-out distance are given in Table 5-16. A support recommendation is given for each class. All this information is transferred to the section plans issued by the survey department.

***Table 5-16 Guidelines used in hazard plan at Colliery 'T'***

| **Guideline** | **Maximum Bord width** | **Maximum Cut-out distance** |
|---|---|---|
| Class A | 6.0m | 9.0m |
| Class B | 6.6m | 18.0m |
| Class C | 7.2m | 24.0m |

A comparative study was done on a total of four borehole drill cores from the No 4 Seam and the results are presented from Table 5-17 to Table 5-20.

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***Table 5-17 Impact splitting results at Colliery 'T', No 4 Seam, borehole G293584***

| Depth (m) | Thickness (cm) | Lithology | Initial Fractures | Final Fractures | Fracture Spacing (cm) | Unit Rating | Weighted Rating | Remarks |
|---|---|---|---|---|---|---|---|---|
| 153 | 23 | S/s | 1 | 8 | 2.9 | 11.5 | -0.6 | Poor |
| 153.2 | 22 | S/s | 1 | 5 | 4.4 | 17.6 | 0.7 | Moderate |
| 153.4 | 20 | S/s | 1 | 3 | 6.7 | 23.3 | 2.8 | Moderate |
| 154.1 | 70 | S | 1 | 2 | 35.0 | 80.0 | 84.0 | Very Good |
| 154.4 | 26 | S | 1 | 2 | 13.0 | 36.0 | 23.8 | Very Good |
| 154.7 | 36 | S | 1 | 4 | 9.0 | 28.0 | 30.6 | Good |
| 155 | 27 | S | 1 | 1 | 27.0 | 64.0 | 64.5 | Very Good |
| | | | | | | | **Final Rating** | |
| | | | | | | | **206** | **Excellent** |

***Table 5-18 Impact splitting results at Colliery 'T', No 4 Seam, borehole G293585***

| Depth (m) | Thickness (cm) | Lithology | Initial Fractures | Final Fractures | Fracture Spacing (cm) | Unit Rating | Weighted Rating | Remarks |
|---|---|---|---|---|---|---|---|---|
| 156.2 | 27 | S/s | 1 | 6 | 4.5 | 18.0 | 0.6 | Moderate |
| 156.4 | 26 | S/s | 1 | 8 | 3.3 | 13.0 | 1.8 | Poor |
| 157.3 | 85 | S | 1 | 6 | 14.2 | 38.3 | 57.0 | Very Good |
| 157.6 | 26 | S | 1 | 4 | 6.5 | 23.0 | 17.6 | Moderate |
| 157.7 | 11 | S | 1 | 1 | 11.0 | 32.0 | 11.6 | Very Good |
| 157.8 | 10 | S | 1 | 1 | 10.0 | 30.0 | 10.5 | Good |
| 158 | 24 | S | 1 | 3 | 8.0 | 26.0 | 23.5 | Moderate |
| | | | | | | | **Final Rating** | |
| | | | | | | | **123** | **Very Good** |

The final rating of 206 from impact splitting classifies the borehole drill core as "Excellent" roof. Final ratings of 123 ("Very Good"), 240 ("Excellent") and 224 ("Excellent") were obtained from the other three impact splitting tests. The results show a good correlation in quantifying the expected roof conditions. Even though the colliery's system did not quantify the roof conditions, the geologist's description of the expected conditions was also a "Good" roof.

**Table 5-19 Impact splitting results at Colliery 'T', No 4 Seam, borehole G293587**

| Depth (m) | Thickness (cm) | Lithology | Initial Fractures | Final Fractures | Fracture Spacing (cm) | Unit Rating | Weighted Rating | Remarks |
|---|---|---|---|---|---|---|---|---|
| 162.2 | 14 | S/s | 1 | 2 | 7.0 | 24.0 | 0.9 | Moderate |
| 163 | 79 | S/s | 1 | 3 | 26.3 | 62.7 | 59.9 | Very Good |
| 163.1 | 10 | S | 1 | 2 | 5.0 | 20.0 | 4.2 | Moderate |
| 163.9 | 80 | S | 1 | 3 | 26.7 | 63.3 | 152.0 | Very Good |
| 164 | 15 | S | 1 | 1 | 15.0 | 40.0 | 23.1 | Very Good |
| | | | | | | | **Final Rating** | |
| | | | | | | | **240** | **Excellent** |

**Table 5-20 Impact splitting results at Colliery 'T', No 4 Seam, borehole G293588**

| Depth (m) | Thickness (cm) | Lithology | Initial Fractures | Final Fractures | Fracture Spacing (cm) | Unit Rating | Weighted Rating | Remarks |
|---|---|---|---|---|---|---|---|---|
| 163.6 | 10 | S | 1 | 1 | 10.0 | 30.0 | 3.3 | Good |
| 163.7 | 10 | S | 1 | 2 | 5.0 | 20.0 | 2.6 | Moderate |
| 163.9 | 10 | S | 1 | 2 | 5.0 | 20.0 | 3.4 | Moderate |
| 163.4 | 30 | S | 1 | 1 | 30.0 | 70.0 | 10.5 | Very Good |
| 163.7 | 30 | S | 1 | 1 | 30.0 | 70.0 | 23.1 | Very Good |
| 163.9 | 20 | S | 1 | 1 | 20.0 | 50.0 | 16.0 | Very Good |
| 164.1 | 30 | S | 1 | 1 | 30.0 | 70.0 | 39.9 | Very Good |
| 164.3 | 20 | S | 1 | 1 | 20.0 | 50.0 | 24.0 | Very Good |
| 164.9 | 60 | S | 1 | 3 | 20.0 | 50.0 | 96.0 | Very Good |
| 165 | 10 | S | 1 | 3 | 3.3 | 13.3 | 5.2 | Poor |
| | | | | | | | **Final Rating** | |
| | | | | | | | **224** | **Excellent** |

### 5.4.4 Colliery 'K'

At Colliery 'K', a roof hazard plan has been developed for the No 4 Seam by rating the roof lithology (e.g. Sandstone) and thickness of coal left in the roof (shown in Figure 5-9) to form a Composite Roof Hazard Plan with the ratings shown in Table 5-21. Due to changes of personnel, the new geologists could not describe how the scores, rating and ranking numbers were obtained. The classifications in Table 5-21 are coloured differently and demarcated in the composite roof hazard plan together with areas of floor roll and sill transgression.

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***Table 5-21 Composite roof hazard plan classification at Colliery 'K'***

| Score | Rating | Rank |
|---|---|---|
| 5 | 21 - 25 | Strong |
| 4 | 16 - 20 | Moderate |
| 3 | 11 - 15 | Weak - Moderate |
| 2 | 6 - 10 | Weak |
| 1 | 1 - 5 | Very Weak |

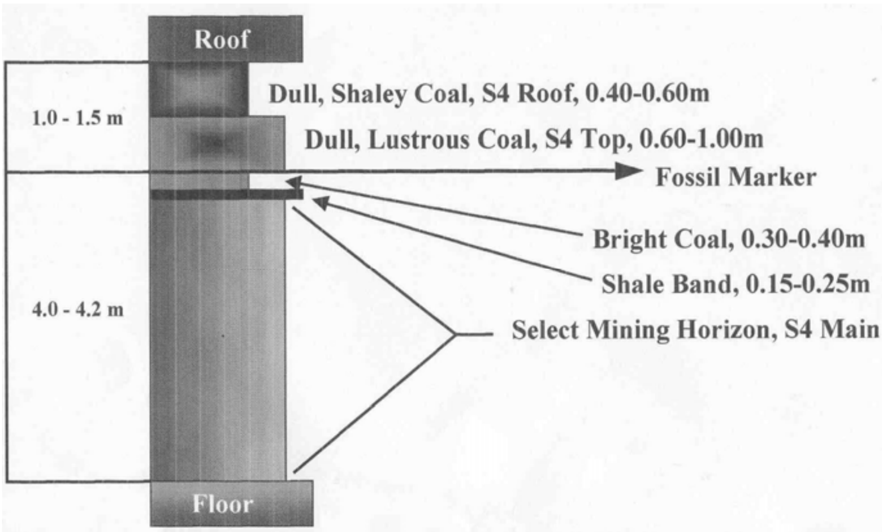


***Figure 5-9 Typical Colliery 'K' No 4 Seam roof lithology***

During this investigation, there was no drilling taking place at Colliery 'K' and thus only one impact splitting test was conducted on borehole drill core KRL3811 and the results are presented in Table 5-22.

When plotted on the composite roof hazard plan, the borehole drill core was on the border of the areas demarcated "Moderate" and "Good". Based on the colliery's system, without underground observations, any of the rankings between Moderate to Weak-Moderate could classify this borehole. However, the impact splitting tests rated it as "Good". The results presented in Table 5-22 are before a coal adjustment factor of 1.56 was applied as explained in the literature review.

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***Table 5-22 Impact splitting results at Colliery 'K', No 4 Seam, borehole KRL3811***

| Depth (m) | Thickness (cm) | Lithology | Initial Fractures | Final Fractures | Fracture Spacing (cm) | Unit Rating | Weighted Rating | Remarks |
|---|---|---|---|---|---|---|---|---|
| 44 | 19.5 | S/f | 1 | 1 | 19.5 | 49.0 | 17.2 | Very Good |
| 44.3 | 25.5 | S/f | 1 | 2 | 12.8 | 35.5 | 21.2 | Very Good |
| 44.5 | 18.5 | C | 1 | 7 | 2.6 | 16.5 | 8.6 | Poor |
| 44.6 | 12.5 | C | 1 | 5 | 2.5 | 15.6 | 6.0 | Poor |
| 44.8 | 20.5 | C | 1 | 7 | 2.9 | 18.3 | 12.7 | Moderate |
| 45 | 20 | C | 1 | 8 | 2.5 | 15.6 | 11.9 | Poor |
| | | | | | | | **Final Rating** | |
| | | | | | | | **78** | **Good** |

### 5.4.5 Colliery 'N'

The Roof Hazard Plan has been established to indicate potential hazards that may affect the safety of the employees. The hazards that are identified are:

- Dykes and sills with associated burnt coal areas
- Laminations, partings and shale from surface
- Sudden change in floor gradient
- All areas of poor roof identified from roof sounding
- Excessive bord widths
- All bord widths exceeding 9 m due to over cutting or scaling
- Horizontal stress concentrations and historical roof fall problems

This plan is constantly revised depending on the identification of new hazardous areas. A separate plan is included in all section plans issued by the survey department. When mining towards an area that has been demarcated in the hazard plan, various procedures come into effect in terms of personnel awareness and roof support. A comparative study was done on Borehole 321, No 4 Seam to test the mines classification of the immediate roof against the results from Impact Slitting Tests. Table 5-23 presents the results of rating of the borehole drill core which has a final rating of 116 (i.e. "Very Good" roof). The geologists also classified the area as good roof on the Roof Hazard Plan.

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***Table 5-23 Impact splitting results at Colliery 'N', No 4 Seam, borehole 321***

| Depth (m) | Thickness (cm) | Lithology | Initial Fractures | Final Fractures | Fracture Spacing (cm) | Unit Rating | Weighted Rating | Remarks |
|---|---|---|---|---|---|---|---|---|
| 229.8 | 66 | S | 1 | 7 | 9.4 | 28.9 | 17.9 | Good |
| 230.2 | 36 | SF | 1 | 10 | 3.6 | 14.4 | 10.6 | Poor |
| 230.6 | 39 | S | 1 | 3 | 13.0 | 36.0 | 39.5 | Very Good |
| 230.8 | 19 | S | 1 | 2 | 9.5 | 29.0 | 18.8 | Good |
| 230.9 | 13 | S | 1 | 1 | 13.0 | 36.0 | 17.2 | Very Good |
| 231 | 10 | S | 1 | 1 | 10.0 | 30.0 | 11.7 | Good |
| | | | | | | | **Final Rating** | |
| | | | | | | | **116** | **Very Good** |

### 5.4.6 Colliery 'S'

The hazard plan used at this colliery is same as that of Colliery 'T'. A comparative study was done on borehole drill core V118043 from the No 4 Seam and the results are presented in Table 5-24 and Table 5-25.

***Table 5-24 Impact splitting results at Colliery 'S', No 4 Seam, Borehole V118043***

| Depth (m) | Thickness (cm) | Lithology | Initial Fractures | Final Fractures | Fracture Spacing (cm) | Unit Rating | Weighted Rating | Remarks |
|---|---|---|---|---|---|---|---|---|
| 84.1 | 15.0 | C | 1 | 5 | 3.0 | 12.0 | 6.9 | Poor |
| 84.2 | 10.0 | C | 1 | 2 | 5.0 | 20.0 | 7.2 | Moderate |
| 84.5 | 29.0 | C | 1 | 4 | 7.3 | 24.5 | 22.8 | Moderate |
| 84.8 | 26.5 | C | 1 | 4 | 6.6 | 23.3 | 16.4 | Moderate |
| 84.9 | 10.0 | C | 1 | 4 | 2.5 | 10.0 | 2.3 | Very Poor |
| 85.0 | 11.0 | SC | 1 | 2 | 5.5 | 21.0 | 4.8 | Moderate |
| 85.1 | 10.0 | SF | 1 | 3 | 3.3 | 13.3 | 2.5 | Poor |
| 85.3 | 19.0 | SF | 1 | 6 | 3.2 | 12.7 | 3.8 | Poor |
| 85.4 | 13.0 | SF | 1 | 2 | 6.5 | 23.0 | 3.8 | Moderate |
| | | | | | | | **Final Rating** | |
| | | | | | | | **70** | **Moderate** |

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***Table 5-25 Impact splitting results, borehole V118043 after coal adjustment factor***

| Depth (m) | Thickness (cm) | Lithology | Initial Fractures | Final Fractures | Fracture Spacing (cm) | Unit Rating | Weighted Rating | Remarks |
|---|---|---|---|---|---|---|---|---|
| 84.1 | 15 | C | 1 | 5 | 3.0 | 18.8 | 3.5 | Moderate |
| 84.2 | 10 | C | 1 | 2 | 5.0 | 31.3 | 4.7 | Good |
| 84.5 | 29 | C | 1 | 4 | 7.3 | 38.3 | 21.2 | Very Good |
| 84.8 | 26.5 | C | 1 | 4 | 6.6 | 36.3 | 24.4 | Very Good |
| 84.9 | 10 | C | 1 | 4 | 2.5 | 15.6 | 4.5 | Poor |
| 85 | 11 | SC | 1 | 2 | 5.5 | 21.0 | 7.1 | Moderate |
| 85.1 | 10 | SF | 1 | 3 | 3.3 | 13.3 | 4.4 | Poor |
| 85.3 | 19 | SF | 1 | 6 | 3.2 | 12.7 | 8.7 | Poor |
| 85.4 | 13 | SF | 1 | 2 | 6.5 | 23.0 | 11.6 | Moderate |
| | | | | | | | **Final Rating** | |
| | | | | | | | **90** | **Moderate** |

A comparative study between the results obtained from impact split tests and mine's roof hazard plan could not be conducted at Colliery 'S' due to unavailability of mine personnel. However, according to mine geologist the expected conditions were moderate.

## 5.5 Application of proactive systems

A series of underground visits also conducted at above given collieries to determine the applicability of the section performance rating and physical risk rating (van Wijk et al., 2002) The results from application of these systems showed that these systems are mine specific and therefore they need to be updated according to different mine standards (e.g. difference in systematic support types and spacing). Furthermore, it was evident that the systems needed someone with a strata control background, as most of the ratings are strata control related and constitute a big weighting in the final rating. The following is a summary of the points to note about the underground section rating systems:

- A structured check list ensured that the user observed and recorded all potential hazards. It also ensures that they are re-evaluated again for improvement in the conditions.
- Different inspectors, i.e. shift supervisors, mine overseers and rock engineers, use the same format. This allows meaningful comparisons to be made in individual sections.
- Systems need to be applied by someone who has a strata control understanding.

- The systems could be used on any mine with small modifications to the control instructions (e.g. support types)
- These systems cannot give a quantification of the required support.

## 5.6 Conclusions and recommendations

The purpose of this task was to evaluate and compare existing roof rating system that are used in South Africa and others that have been developed in other countries, and proposing the way forward for the development of a system that could be used universally on South African collieries to determine the roof conditions and quantitatively required support. The results showed that although many collieries have hazard plans, these plans do not readily quantify the mechanistic behaviour of the roof strata, they are mostly descriptive and are subject to different opinions. Therefore, they cannot be used for roof support design purposes. Furthermore, there is no uniform methodology behind the development of these plans, which makes it difficult for another person to apply them.

The CMRR could overcome most problems associated with the application of rock mass classification systems to coal mining. Also, in principal, the borehole core CMRR is a very similar system to impact splitter. However, due to its origin from case histories from the United States, certain modifications need to be applied to the system for the different conditions in South African coal mines. In the context of the South African coal mining industry, the following summary can be drawn regarding future improvements in the system:

- Requires exposure into the roof (underground CMRR only)
- Only the bolted height is rated. In South Africa, 2.0 m into the roof is the height that is usually rated.
- Although sets of joints have been considered in CMRR, single joints can have an influence and should thus also be included.
- Joint orientation is not included (underground CMRR only).
- Stress adjustment is required in the rating system to account for the influence of high horizontal stress (underground CMRR only)
- Blasting adjustment is not considered (underground CMRR only)
- Does not consider the position of soft or hard layers into the roof (both underground and borehole core CMRR)
- Requires skilled personnel to carry out ratings (both underground and borehole core CMRR)

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Rating systems will continue to play an important role in coal mining practice. These systems should relate to the expected mode of failure of the strata for design and planning purposes. Underground rating and performance systems need to be incorporated with the roof rating systems into the overall ground control management to ensure adherence to design and overall mine standards. However, these systems cannot quantitatively determine the required support system in a given condition.

Although most collieries studied had some form of hazard identification systems in place, these systems are mostly descriptive in nature and therefore tend to be subjective. Moreover, these rating systems are used mainly for planning purposes, and not to determine the changing conditions underground. The systems have worked in some cases where one person had extensive experience at one mine. However, due to movement of personnel, there has been a loss of knowledge, insufficient documentation and a lack of updates of the local systems.

Impact splitting test has been found to be an appropriate system to eliminate human error in core rating. The advantage of impact splitting over the individual colliery‘s geology based rating systems is its ability to readily quantify the roof instead of a mere description that can change from one person to another. Geology based systems have been developed from experience by mine personnel that certain soft or hard layers in the roof were a major cause of instability. During this study, impact splitting has shown a very good correlation with the geology based rating systems. The system can therefore be used during planning for good prediction of conditions ahead of mining. Furthermore, the system requires minimal training time and therefore does not require skilled personnel.

In conclusion, impact splitting tests, section performance rating and physical risk ratings systems developed in South Africa can be described as the most effective and appropriate for South African conditions. Impact splitting can readily quantify the roof conditions during planning with minimum subjectivity. Section performance and physical risk rating can be used for identifying changing conditions while mining and determining the best response to the different conditions.

It must however be noted that as shown in the previous chapters of this thesis that the roof lithology, stress regime and roof characteristics can change within meters in a production section. Therefore, in order to predict these changing conditions many boreholes are required for a section, which would be very expensive and time consuming. In addition, borehole core based systems like the impact splitting are dependent on the quality of the core. Layers that are very weak or have very low cohesion can easily break during the drilling process. Geophysical techniques may therefore be more accurate in such cases for identification of these layers.

# Chapter 6.0

# Evaluation of roof bolting systems in South Africa

## 6.1 Introduction

One other important consideration in determining the performance of a support system as a whole is the bolting components that are used in the design. Therefore, an in-depth study into the bolting elements that are currently being used n South African collieries was conducted.

There are five important components of a bolting system, which determine the quality of an installed support. These are:

- Machinery; equipment;
- Bolt;
- Resin;
- Hole;
- Rock type

These five components are of equal importance, as failure of any of these will result in an inadequate support system. Therefore, as part of this task, all important parameters of these five components have been investigated. The important parameters of the five components are given below:

**Machinery and equipment**

- Torque;
- Thrust;
- Effect of different drill bits on the support performance; and
- Free rotation, spinning and drilling speeds.

**Bolt and components (thread, nut and washer)**

- Bolt profile;
- Effect of preload on bolt and components;
- Variation of diameter and rib heights; and
- Deformability.

**Resin**

- Set and spin times;
- Effect of roofbolter spinning speed;
- Resin type; and
- Effect of plastic encapsulation.

**Bolt hole**

- Effect of wet and dry drilling on system performance and hole profile;
- Hole profile as a function of the bit characteristics;
- Size of annulus between bolt and hole;
- Effect of drilling speed on hole profile.

**Rock type**

The geology is also a very important external component of the support system. An understanding of the interaction between the rock and the bolting system is crucial; therefore, to achieving the most appropriate support system for different geological environments.

## 6.2 Specifications for roofbolters

### 6.2.1 Introduction

The quality of installation of a support system is directly related to the performance of the equipment that is used to install the bolts. The performance of bolting equipment was therefore investigated as part of this study in order that the relative importance of the various machine parameters could be ascertained, as well as the range in values of these parameters as provided by the equipment used in South African collieries.

The following parameters were assessed in determining the performances of bolting equipment:

- Drilling speed: determines the hole profile;
- Spinning speed: determine the resin mixing characteristics and hole profile;
- Torque: determines the tension on the bolt and the capability of installing shear-pin bolts;
- Thrust: determines the hole profile and pushing the bolt through the resin; and

These parameters were then measured against roof bolt performance in various rock types. It should be noted that currently in South Africa, there are no standards for these parameters in collieries, except the torque, which should be approximately 240 Nm (Torqueleader, 2005) in order to generate approximately 50 kN (5 tonnes) for tensioning by roofbolters.

A total of 143 roofbolters, which were operational during the evaluation, were tested from 27 different collieries, ranging from Tshikondeni in the north to Zululand Anthracite Colliery (ZAC) in the east. This provided a comprehensive database of roofbolter information. Tests were done on a variety of machines from different manufacturers, including Rham, Fletcher, Voest Alpine, License, Klockner, Biz Africa, along with custom-designed bolters manufactured by particular mines. Results from all of these machines varied widely, even to the extent of differing from boom to boom on twin boom machines.

### 6.2.2 Testing procedure

During this investigation, the testing procedure for each machine followed a set pattern, which was developed to be as quick and easy as possible, in this way minimizing any possible downtime to production machines. For each machine, the torque setting at which the machine spins the bolt was measured, to ensure that the machine was capable of breaking out either the crimp or shear pin of the bolt, if such a future was present.

Following this, a hole was drilled and the speed of drilling was measured in revolutions per minute using a laser digital tachometer. This device quickly and easily measures the speed by simply attaching a reflective strip to the drill chuck or drill steel, and shining the laser onto the strip while the drilling is in progress.

Once the hole was drilled, the depth was measured and a borehole micrometer was inserted to measure the hole diameter at intervals along the length of the hole. This gives an indication of the hole profile as drilled by the particular bit type at a specific rotation speed. Measurements were taken from two to three holes per roofbolter.

A bolt is then inserted into the chuck and a load cell fitted over the bolt. The bolt is pushed into the hole, without inserting resin, and pushed against the roof with the maximum force possible to establish the thrust that the roofbolter is capable of exerting against the bolt, which is important when full-column roof bolts are being installed and a bolt is being pushed through several resin capsules.

The bolt was then installed with resin and a speed measurement is taken while the bolt was being spun through the resin. This measurement shows the speed at which the resin is being mixed.

The form, presented in Figure 6-1 was used to record measurements during the testing. Other measurements taken were standard lengths and diameters, the bit type and diameter, drill steel length and diameter, type of bolt, bolt length and diameter. The type of support, be it mechanical point anchor, resin point anchor or full-column resin was noted and resin type, capsule length and diameter recorded.

Finally, drilling type (wet or dry) was noted, as this may have considerable impact on the hole profile in different rock types. Where possible, a borehole log of the area in which tests were conducted was collected in order to take into account the influence of the immediate roof in which installation is taking place.

| | | |
|---|---|---|
| **Date** | 29/11/2002 | |
| **Mine** | Goedehoop - Hope Shaft | |
| **Section** | 9/10 | |
| **Mining Method** | Bord and Pillar - CM | |
| **Production Rate** | +/- 1000 tonnes/shift | |
| **Type of Roofbolter** | Fletcher MDDR - 17 SN - 2001026 | |
| **Date of Purchase** | 14/06/2001 | |
| **Cycle Time (Bolts per hour/shift)** | LHS - 60 seconds / 1.5m RHS - 77 seconds / 1.5m | |
| **Bit Type** | Spade | |
| **Bit Diameter (mm)** | 25.3mm | |
| **Drill Steel Diameter (mm)** | 22.3mm - Flat 24.1mm - Hex | |
| **Drill Steel Length (m)** | 1.44m | |
| **Type of Support** | f/c | |
| **Type of Resin** | Fasloc 'A' spin to stall | |
| **Capsule Diameter (mm)** | 21.4mm | |
| **Capsule Length (mm)** | 495mm | |
| **Type of Bolt** | Rebar, shear pin | |
| **Bolt Diameter (mm)** | Core - 20.2mm Rib - 21.3mm Parallel - 20.4mm | |
| **Bolt Length (m)** | 1.5m | |
| **Bolt Consumption** | +/- 80 / shift | |
| **Washer Type** | Dome - Dog Eared | |
| **Washer Dimensions (mm)** | 125 x 125 x 5.1mm | |
| **Type of Pin/Nut** | Shear pin | |
| **Dry/Wet Drilling?** | Dry | |
| | **Left Boom** | **Right Boom** |
| **Free Rotation Speed (rpm)** | 614 | 622 |
| **Drill Speed (rpm)** | 605 | 572 |
| **Resin Spinning Speed (rpm)** | 604 | 592 |
| **Torque (Nm)** | 180 | 260 |
| **Thrust (kN)** | 780 | 500 |
| **Hole Length (m)** | 1.44m | 1.45m |
| **Borehole Log** | | / |

| **Hole Profile** | | |
|---|---|---|
| **1** | **2** | **3** |
| | | |
| | | |
| | | |
| | | |
| | | |
| **4** | **5** | **6** |
| | | |
| | | |
| | | |
| | | |
| | | |

1. The first hole profile reading should be taken +/- 2 inches from the back of the hole.
2. Bolt should be pushed through the resin before measuring spinning speed.
3. Three speeds are to be measured. Free rotation, drilling, and resin spinning speeds.
4. Stop measuring the drilling speed before the hole is finished.
5. Bolt diameter measured across core, across ribs and across parallel rib.

***Figure 6-1 Form used for recording data from equipment tests***

### 6.2.3 Results

#### 6.2.3.1 Rotation speed during drilling

The results of rotation speed during drilling are presented in Figure 6-2, Figure 6-3, Figure 6-4, and Figure 6-5. These figures highlight that there is a significant variation in the drilling speeds of various bolters. As would be expected, the curve is shifted lower down the axis with the introduction of load to the system. The maximum rpm is 816, with a minimum of 148 rpm. Results for Bolter B are above the average, the largest proportion being in the 550 to 600 rpm range. Similarly, Bolter A and other bolters behave in the same way as the majority of the results falling within the 250 to 400 rpm range. The effect that rock type has on the drilling speed is discussed later in the report.

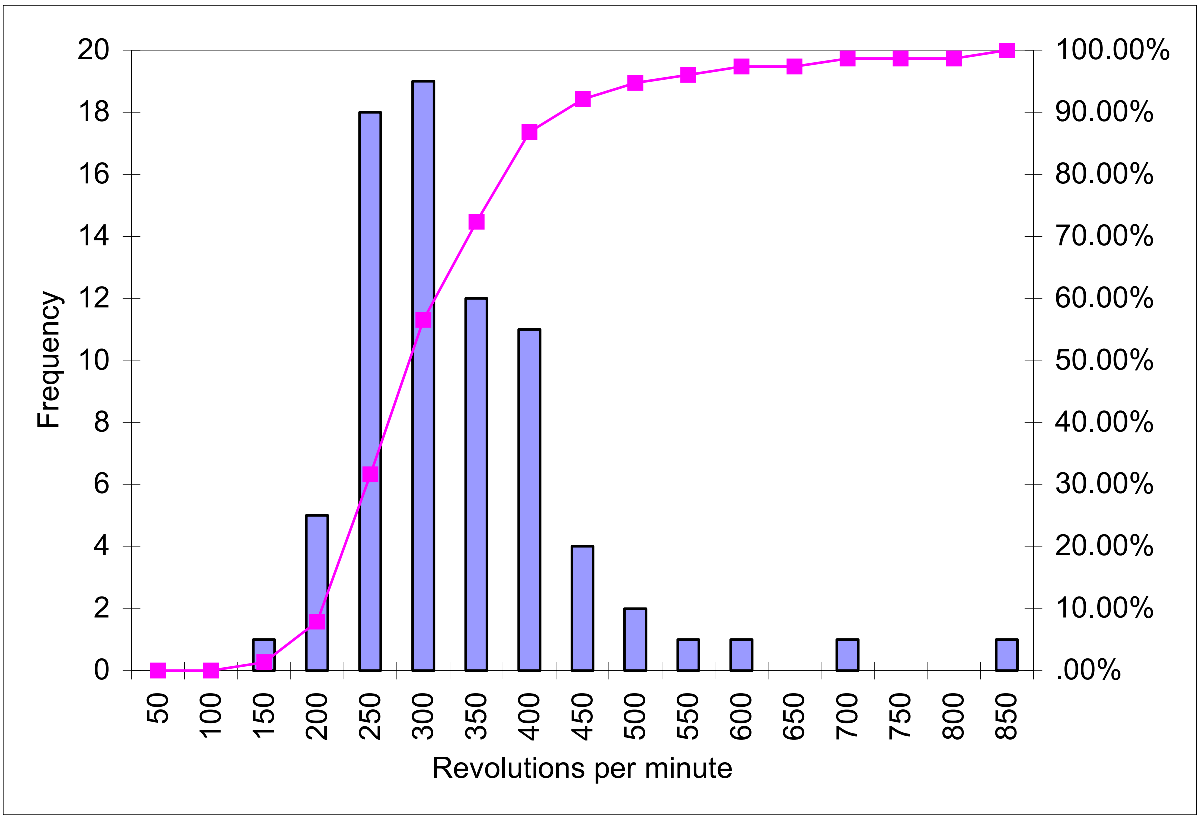


***Figure 6-2 Drilling speed - bolter A***

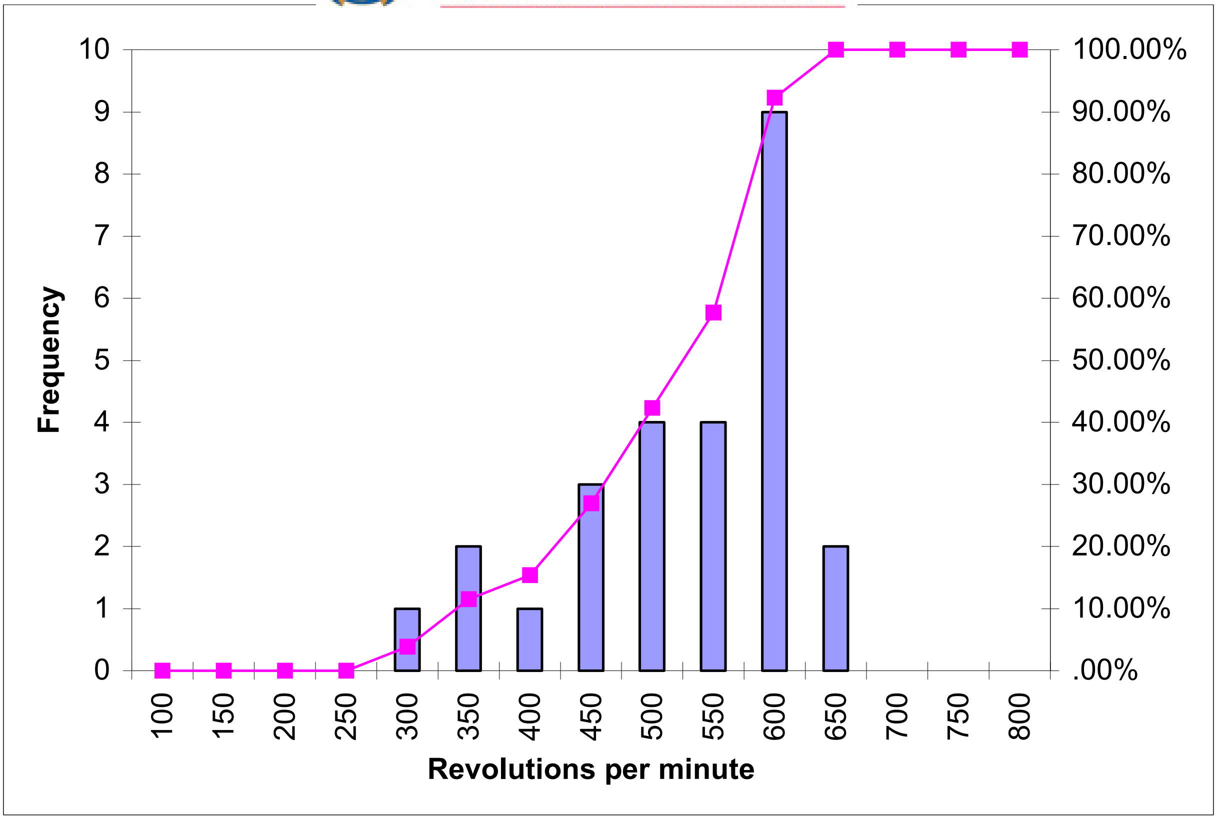


*Figure 6-3 Drilling speed - bolter B*

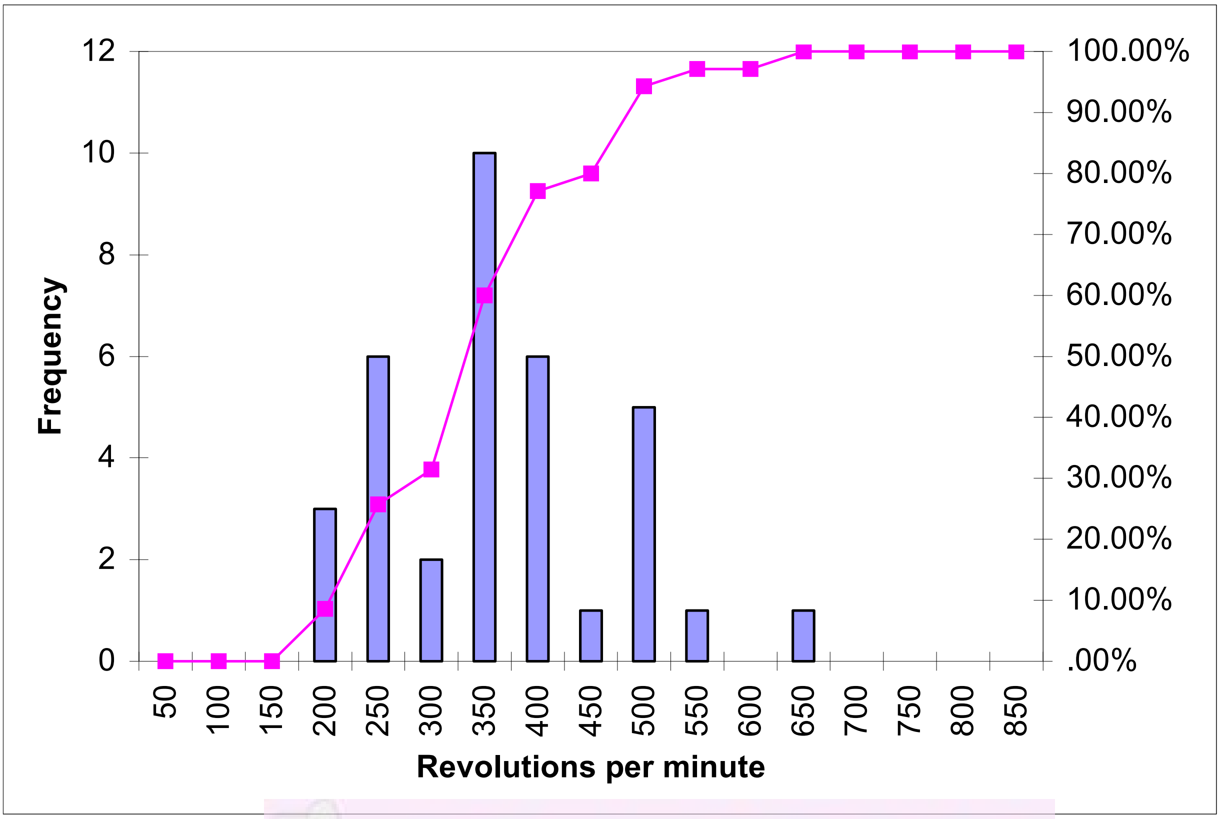


*Figure 6-4 Drilling speed - other bolters*

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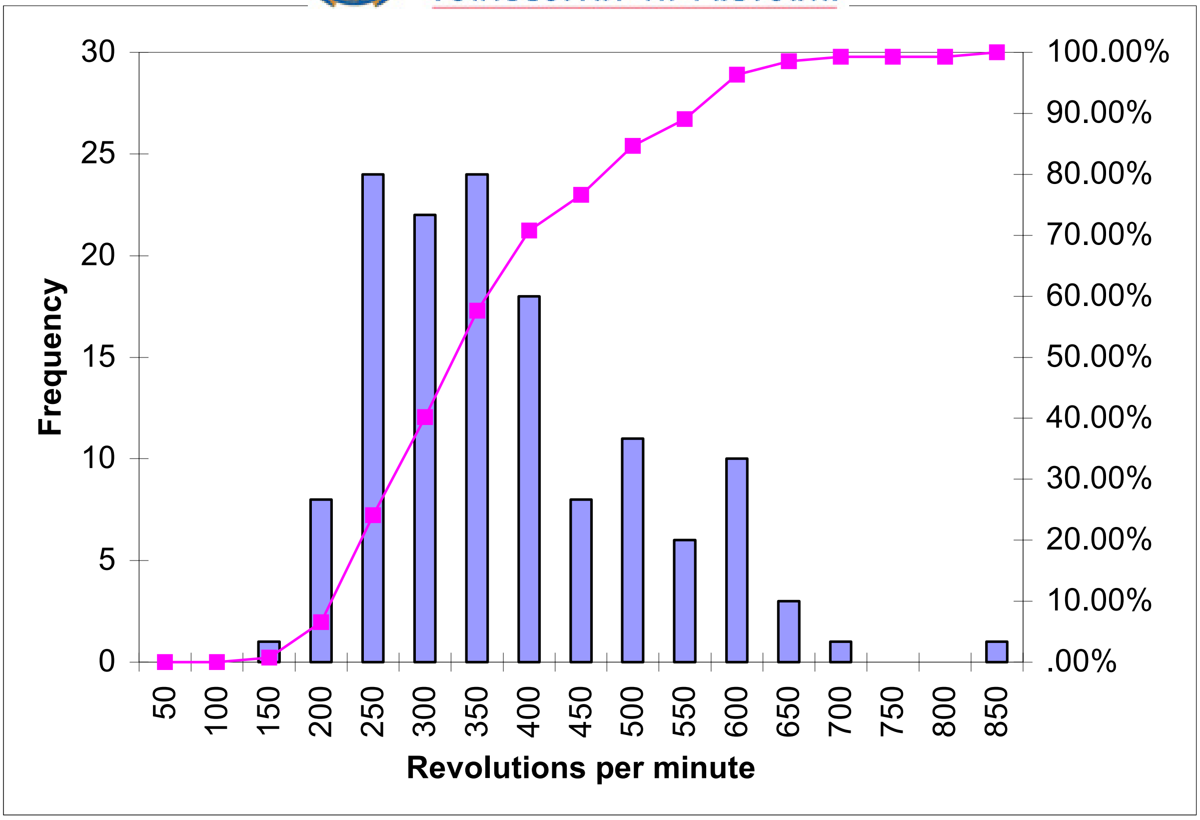


***Figure 6-5 Drilling speed - all bolters***

6.2.3.2 Resin spinning speed

The speeds measured while spinning resin for various types of bolters are shown in Figure 6-6, Figure 6-7, Figure 6-8, and data from all the bolters is plotted in Figure 6-9. Resin spinning speeds, generally, show much lower results than either of the other speed measurements. The resistance offered by the resin capsule in a confined space reduces the speed considerably. Resin spinning speed shows a maximum speed of 643 rpm and a minimum of 45 rpm. The distributions within the groups, however, tend to be similar to drilling speed, with the results of Bolter B being proportionately higher than those of the other two groups. Resin manufacturers recommend a spinning speed of between 400 and 500 rpm on “A” type spin-to-stall resin. Obviously, too low a spinning speed may not mix the resin correctly in the required spinning time, and result in a weak bond. It is also possible that too high a spinning speed may over-spin the resin, damaging the bond and reducing the strength. Figure 6-9 indicates that the resin spinning speeds of approximately 22 per cent of all bolters tested are within the resin manufacturers recommended range.

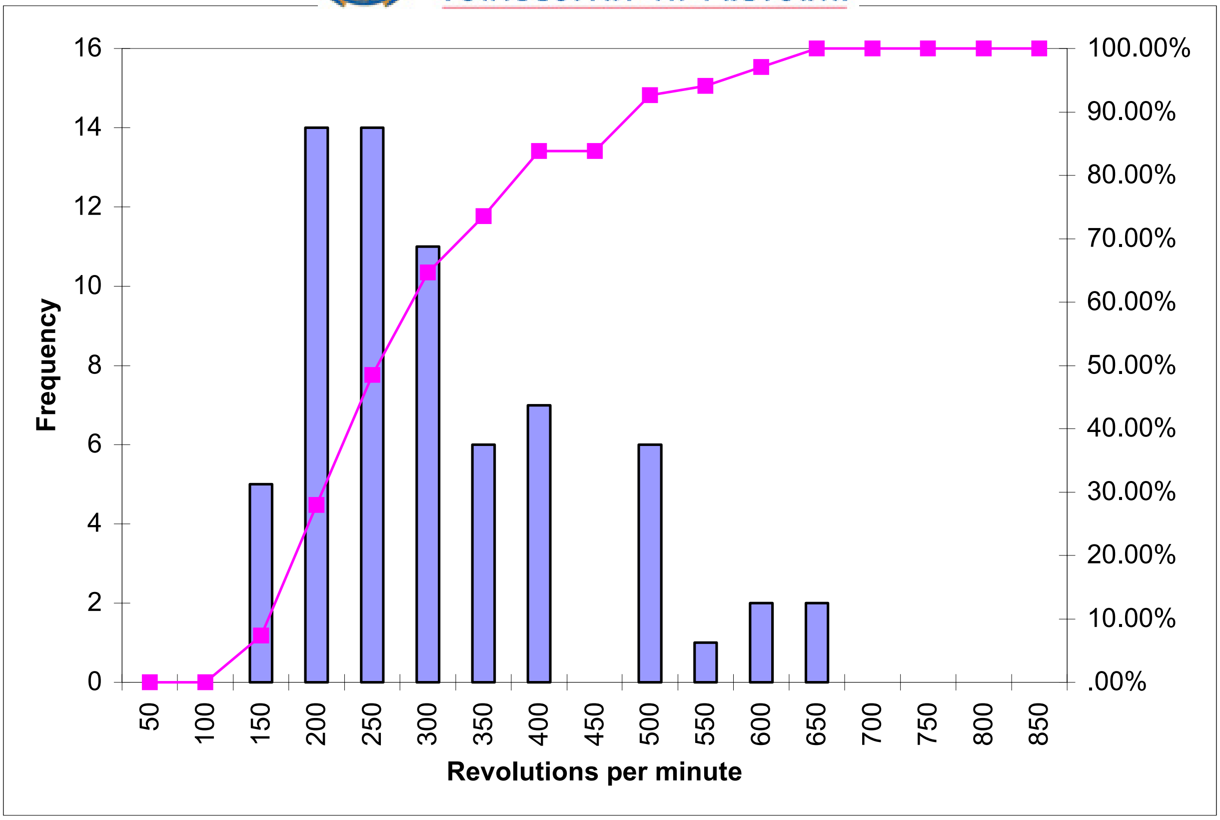


*Figure 6-6 Resin spinning speed - bolter A*

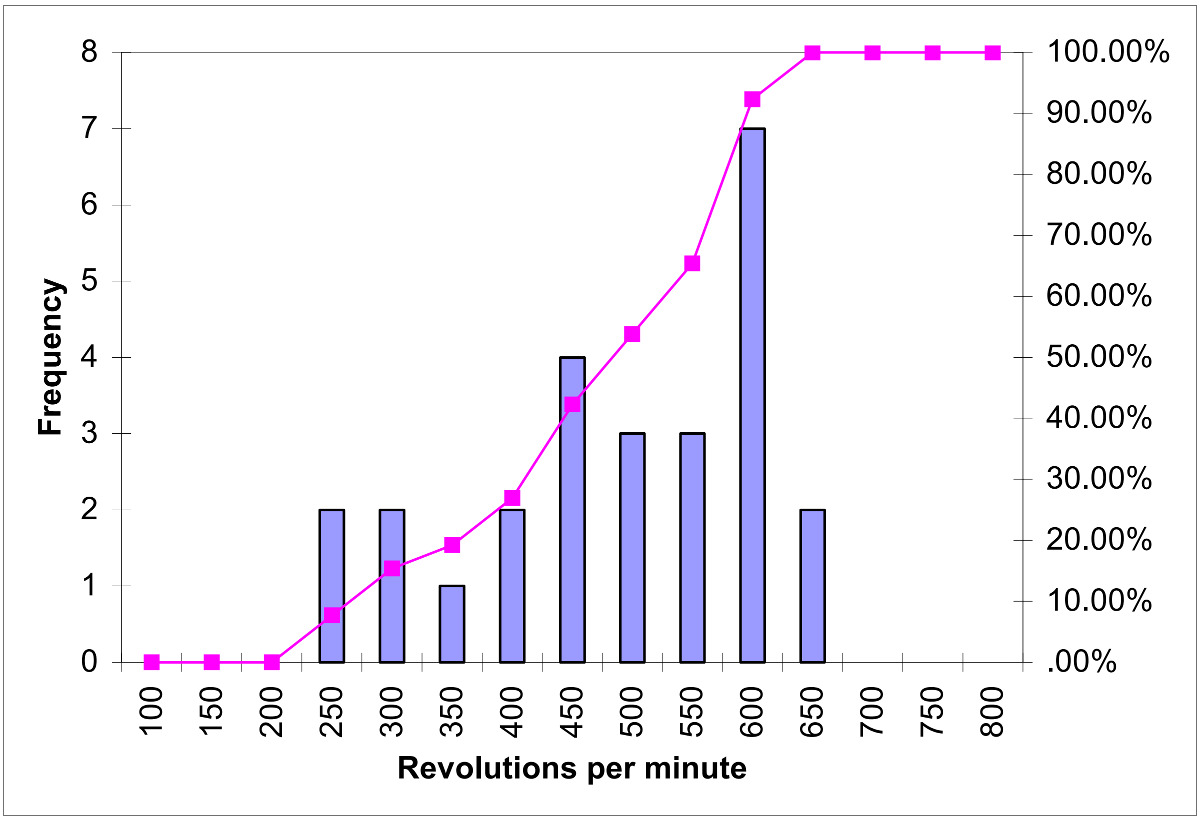


*Figure 6-7 Resin spinning speed - bolter B*

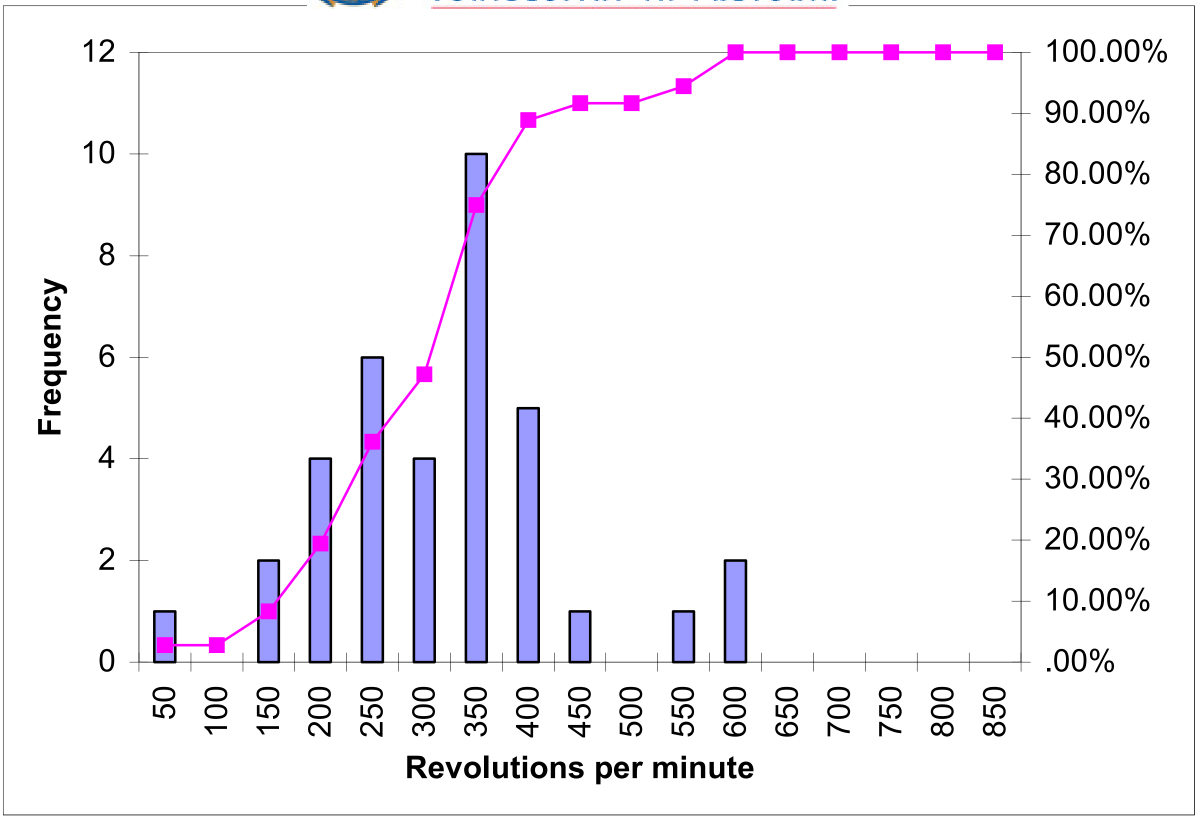


***Figure 6-8 Resin spinning speed - other bolters***

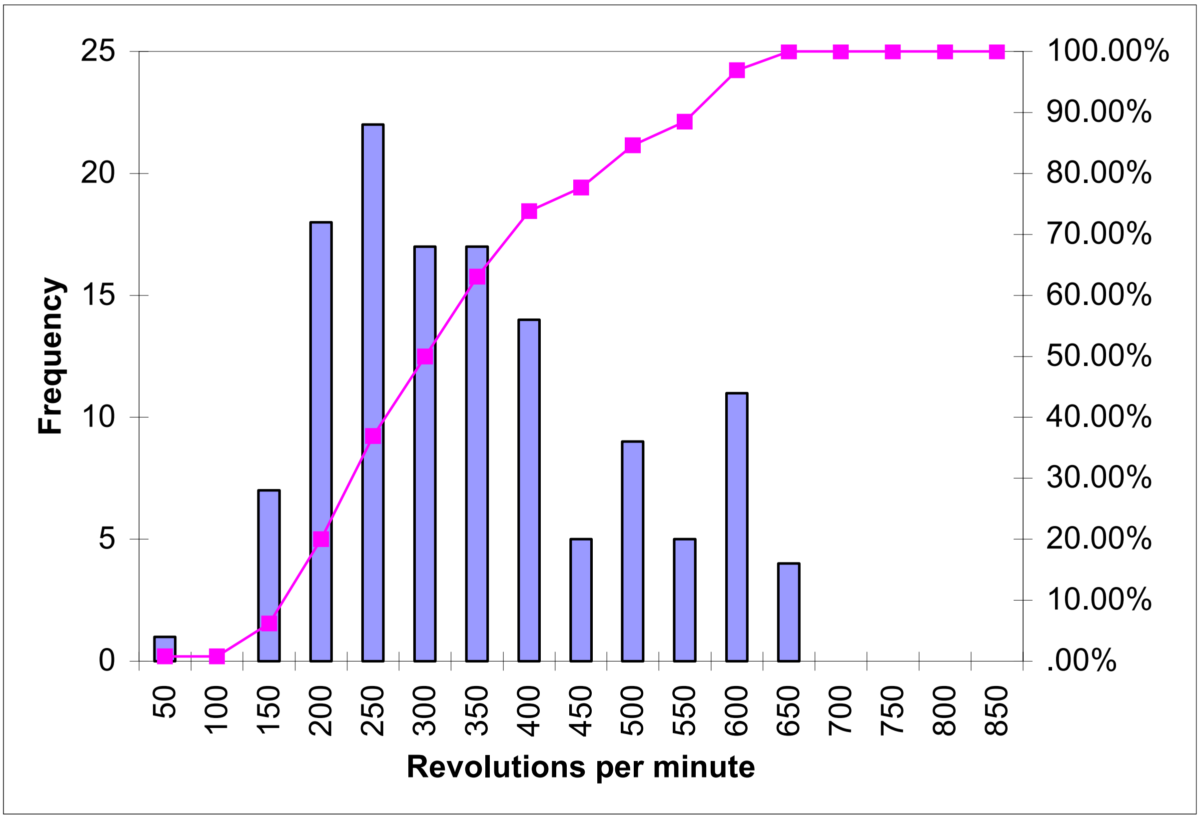


***Figure 6-9 Resin spinning speed - all bolters***

#### 6.2.3.3 Torque

Currently in South Africa, a roofbolter is expected to produce 200 Nm to 250 Nm torque at all times in order to tension the bolt to approximately 50 kN (5 tonnes).

In the drilling phase, enough torque is required to allow the bit to penetrate whatever rock type may be present in the roof and pass through harder layers with the same efficiency as through soft. When the bolt is installed, enough torque is also required to ensure a sufficient mix of resin and catalyst and also to break out the crimp or shear pin on a bolt, should one be present.

The results from the torque measurements are shown in Figure 6-10, Figure 6-11, Figure 6-12, and Figure 6-13. These figures indicate that the torque on all machines ranges from a maximum of 560 Nm to a minimum of 50 Nm. The lower value is not sufficient to break the crimp or shear pin (120 kN torque is required to break the shear pin), and this was observed to be the case on one mine. The bolter in question was tested and found to provide torque of 80 Nm. Observation of the roof bolt crew trying to install bolts made it clear that the machine was unable to break out the shear pin. The spread of torque values for all bolters show a similar distribution and variability.

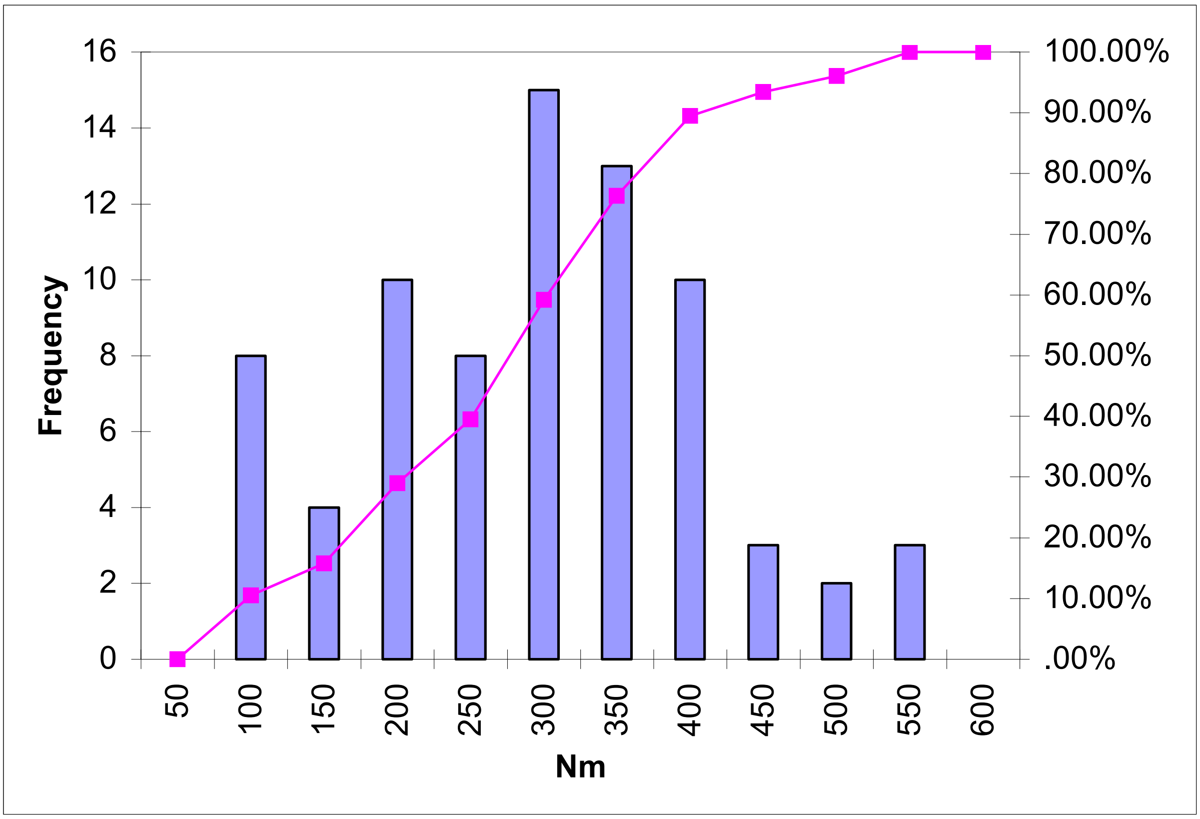


***Figure 6-10 Torque - bolter A***

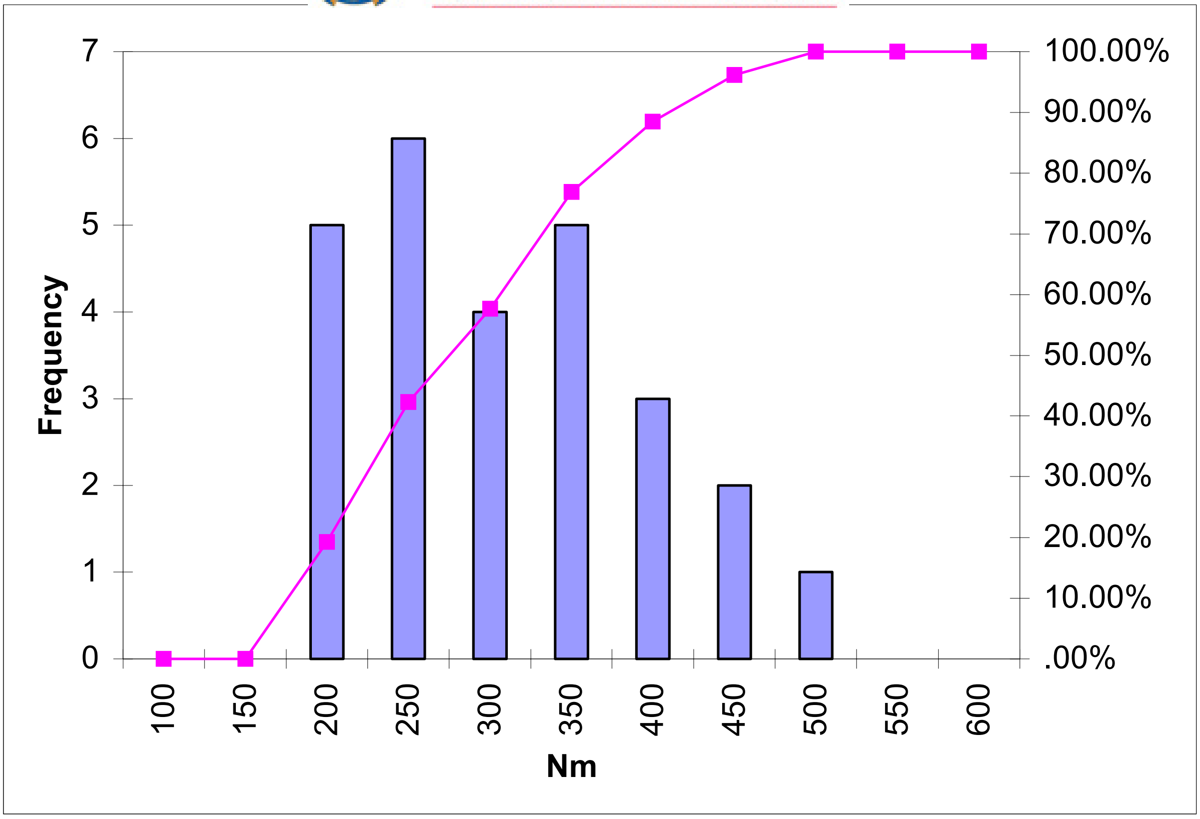


*Figure 6-11 Torque - bolter B*

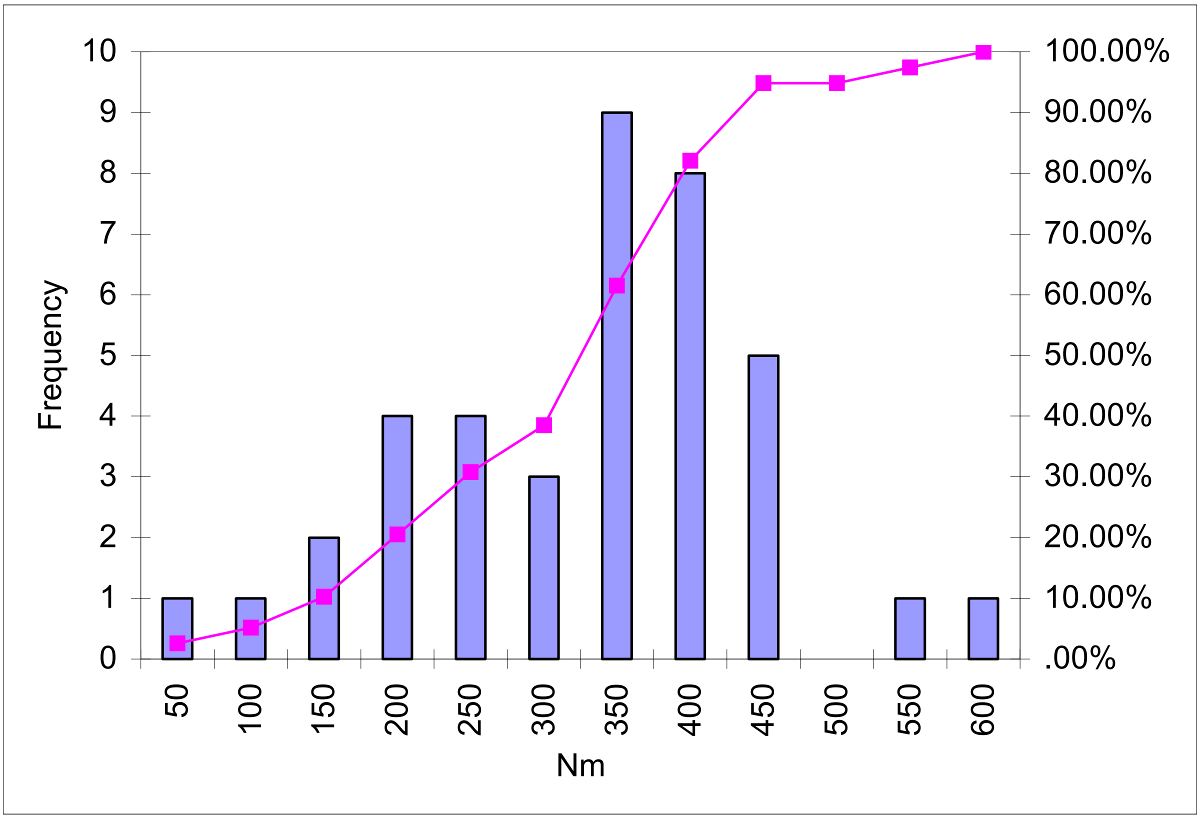


*Figure 6-12 Torque - other bolters*

Figure 6-13 indicates that only 26 per cent of all bolters had torques within the 200 Nm to 250 Nm range.

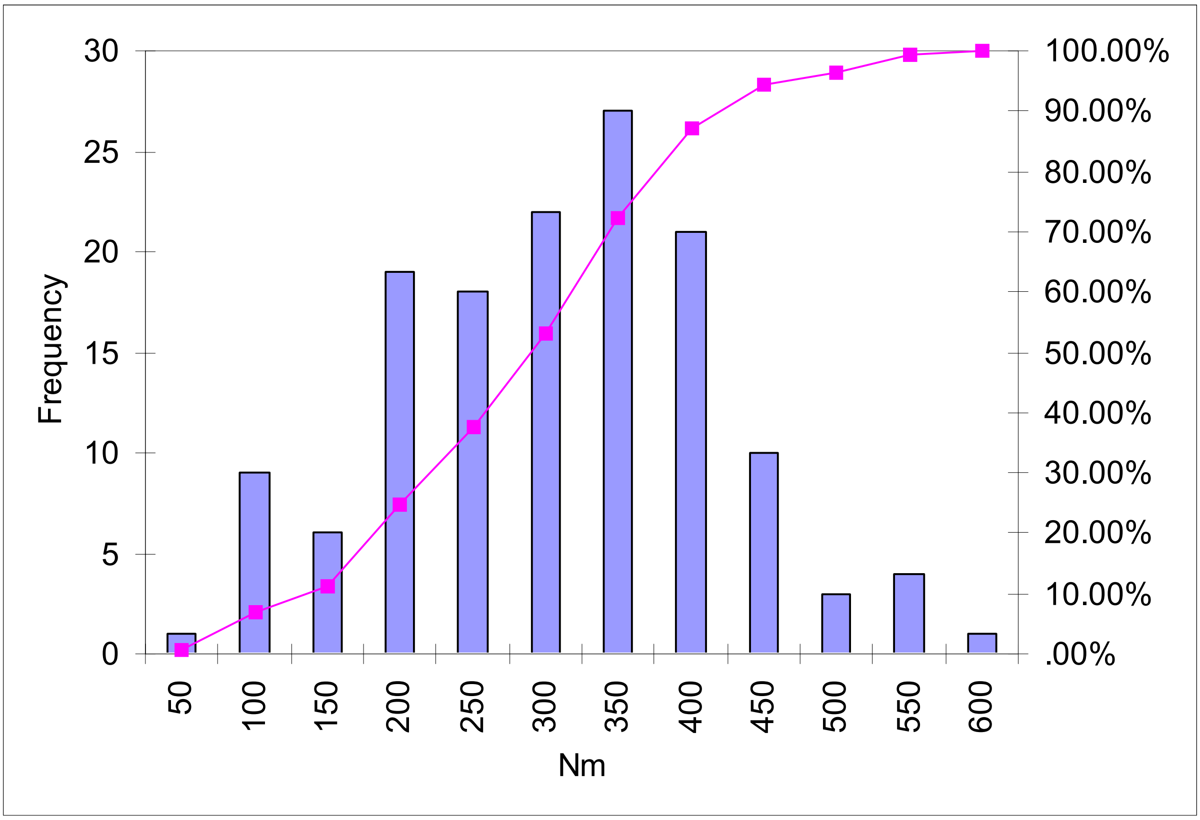


***Figure 6-13 Torque - all bolters***

6.2.3.4 Thrust

Thrust is the axial force exerted on the drill steel by the machine. Thrust applied while a hole is being drilled is difficult to measure. For this reason, the thrust given in this section is the maximum thrust capacity of the machine. Thrust is required in order to penetrate the roof, and also to force the bolt through a resin capsule to the back of the hole before spinning occurs. Thrusts of the roofbolters tested vary significantly, from as little as 10 kN to 32 kN, with an average of around 18 kN.

The results are presented in Figure 6-14, Figure 6-15, Figure 6-16, and Figure 6-17.

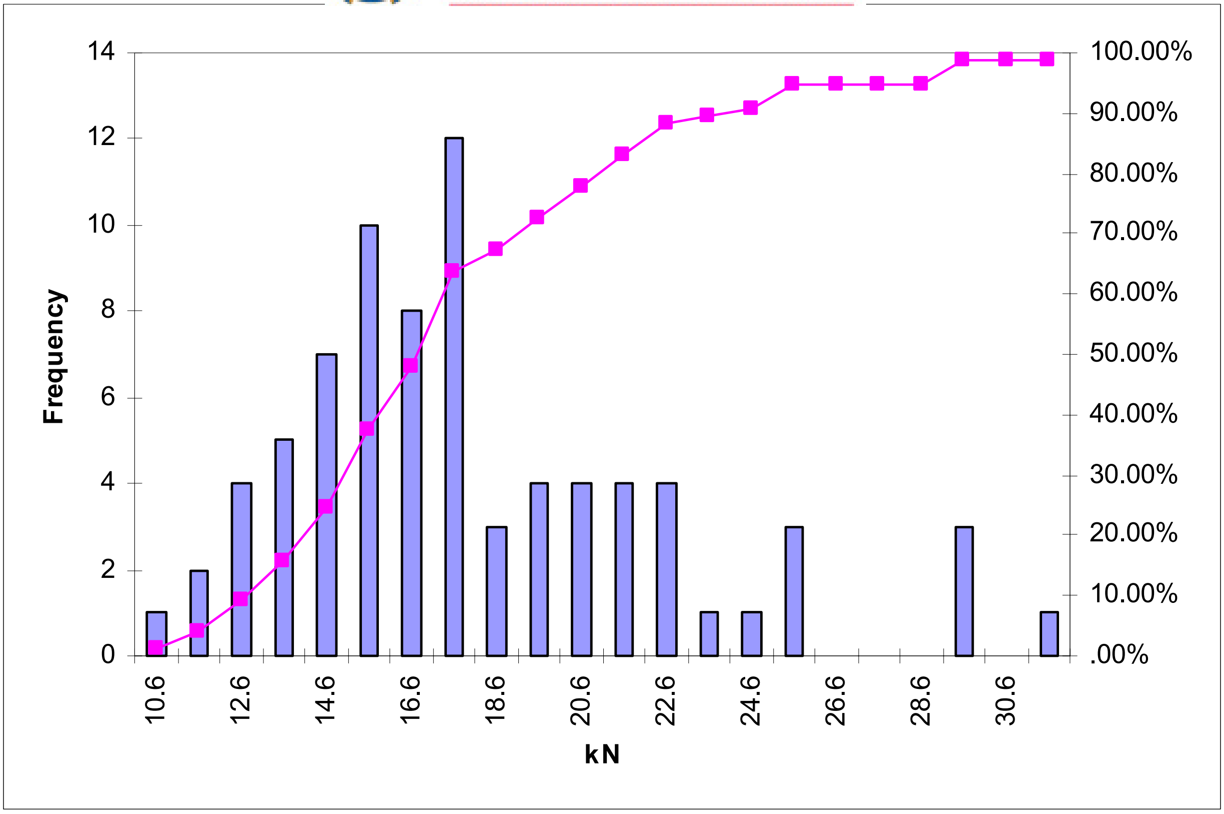


***Figure 6-14 Thrust - bolter A***

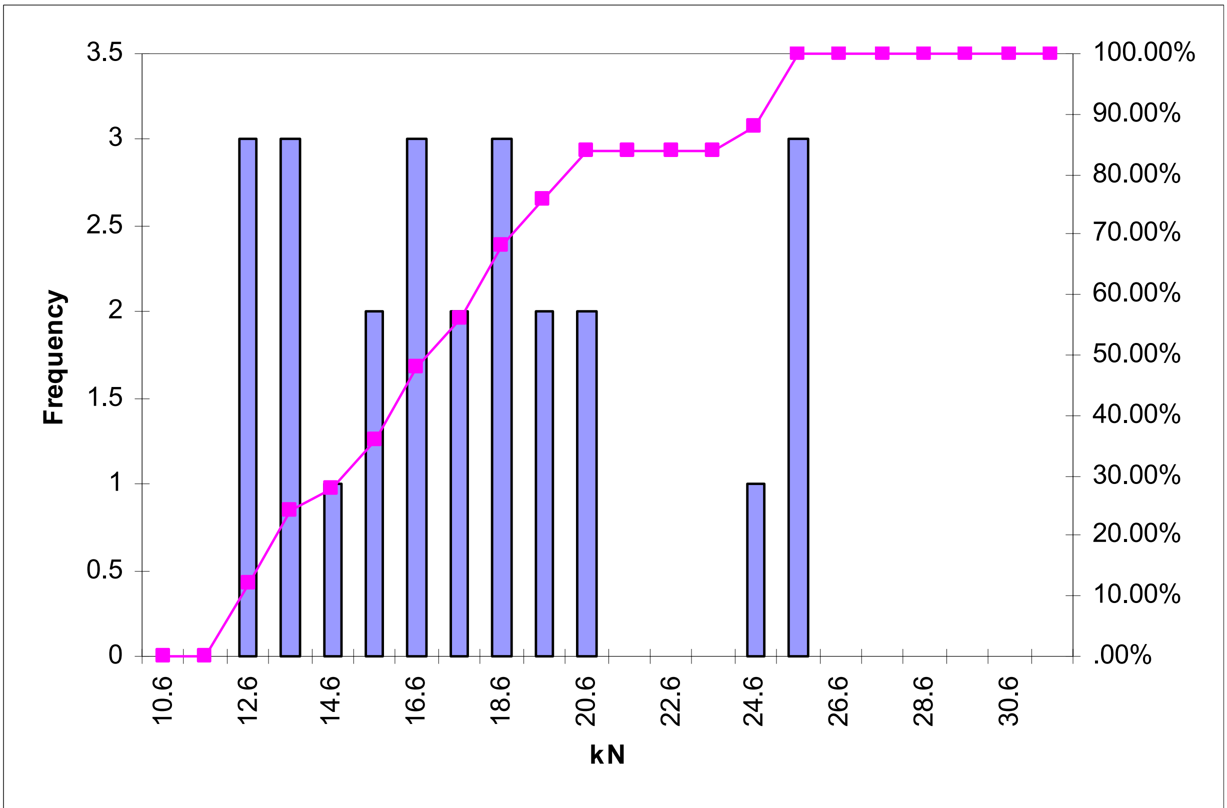


***Figure 6-15 Thrust - bolter B***

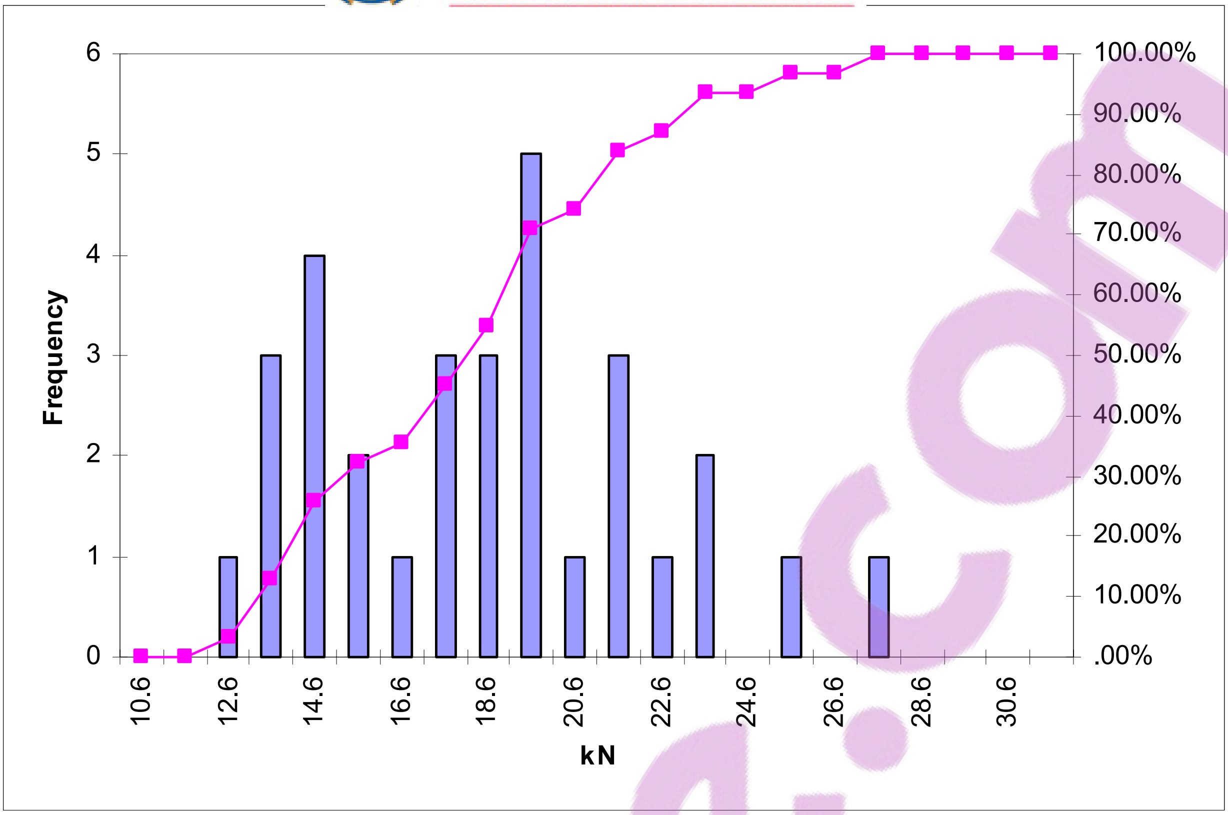


*Figure 6-16 Thrust - other bolters*

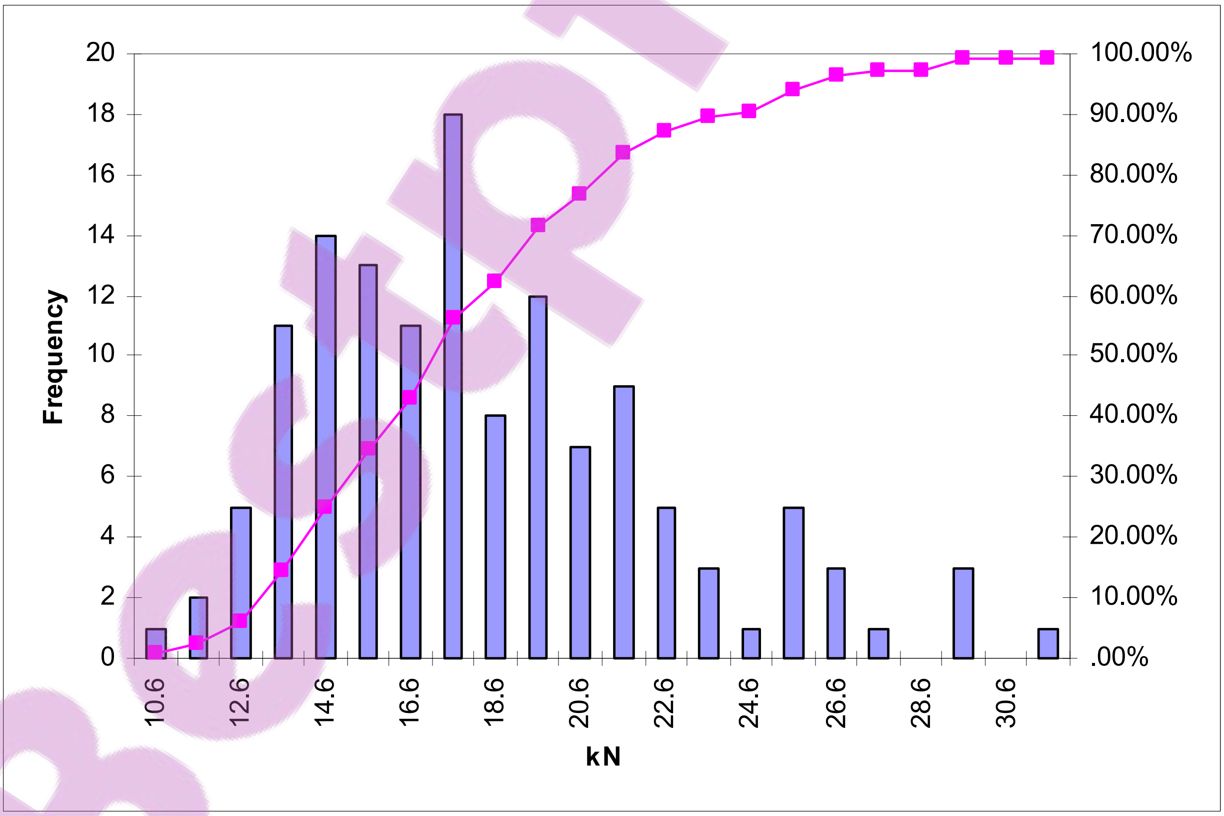


*Figure 6-17 Thrust - all bolters*

#### 6.2.3.5 Hole profile

The hole profile is also a fundamentally important parameter, as it determines the bonding quality between the resin and rock. A smooth-walled hole would exhibit lower bond strength than a hole wall that is serrated or ‘rifled’.

Currently, there is no suitable tool available to determine the hole profile, apart from overcoring. However, overcoring is very expensive and cannot practically be used for a large-scale experiment applied to all available bolters in South Africa.

Therefore, the hole profile is measured by taking a number of hole diameter measurements at regular intervals along the hole. This gives an indication of the quality of hole being drilled in each particular test. A mean is calculated for the five diameter measurements, and the standard deviation determined. The standard deviation gives a description of the quality of hole drilled; the smaller the deviation, the smoother the hole. With this in mind, comparisons were made between hole quality and other measurements in an attempt to try and find links between the controllable factors and the quality of hole. The most obvious factors influencing the hole quality should be the drilling speed, torque and thrust of the bolter in a particular rock type. As can be seen from the graphs below, no correlation was however found between any one of these factors. The hole profile was then compared for wet drilling and dry drilling machines, again results indicates no apparent correlation.

As shown in Figure 6-18, the largest percentage (approximately 80 per cent) of standard deviation on all holes, drilled by all machines, in all different roof types, is less than 1.0 mm diameter over the entire hole length. Although 1.0 mm may seem insignificant, the fact remains that most 25 mm drill bits are shown to be drilling 27 to 28 mm diameter holes. This indicates that most 20 mm bolts are being installed in a hole with an annulus of up to 10 mm, when the worst case example of almost 2 mm standard deviation is taken.

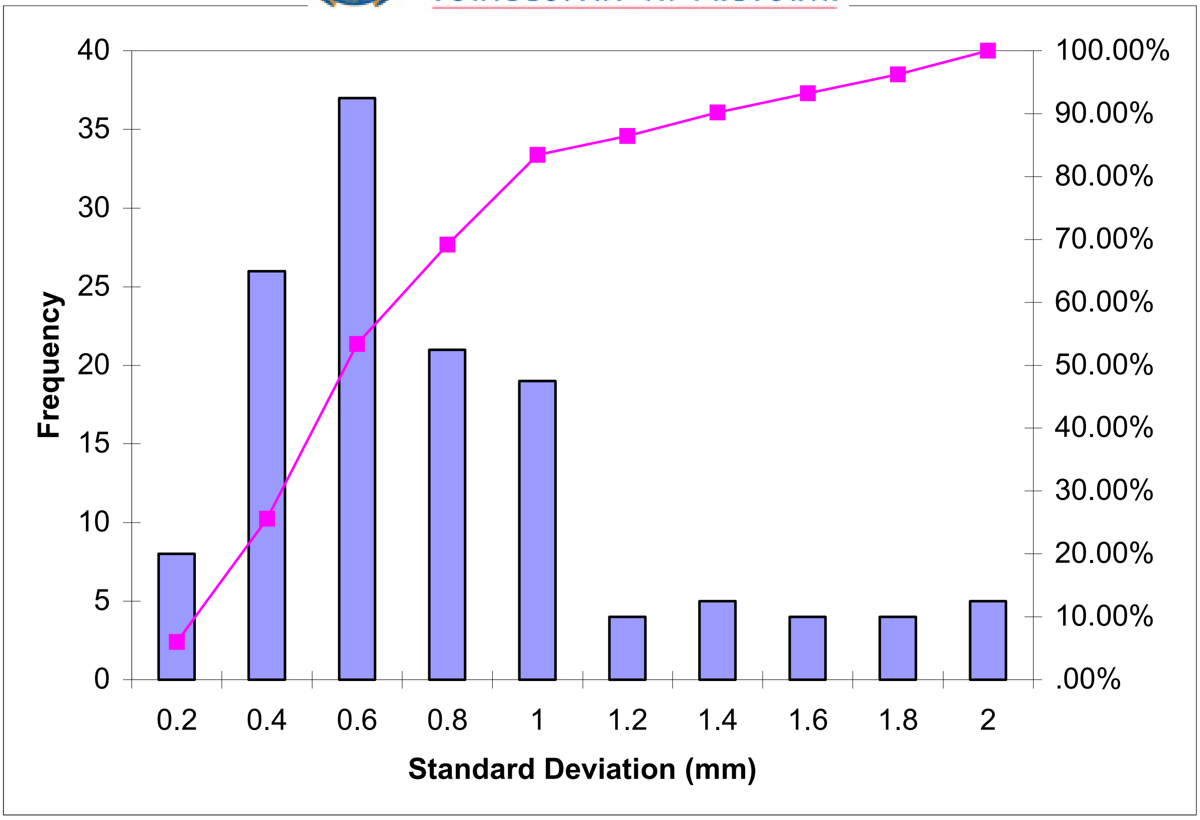


***Figure 6-18 Hole profile standard deviation frequency***

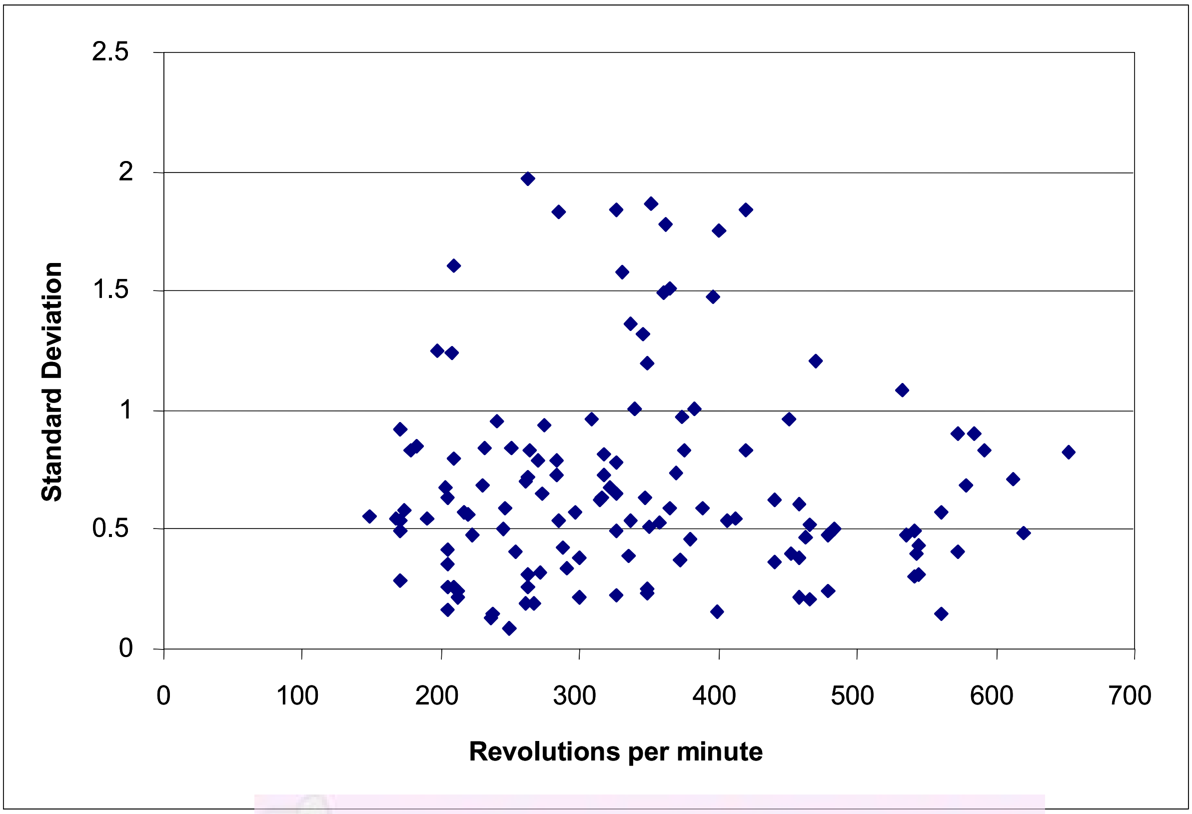


***Figure 6-19 Drilling speed against hole profile standard deviation***

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One of the most obvious factors influencing the quality of the hole would be the speed at which the hole is drilled. A hole drilled at high speed would either have a very smooth profile as a result of the speed of drilling, or would produce a large diameter hole because of inadequate flushing, which is more likely at high speed. As can be seen from Figure 6-19, there is no correlation between drilling speed and hole diameter standard deviation.

Figure 6-20 shows that there is a very wide range of torque settings on roof bolting machines in South Africa, and that they do not correlate with the regularity of the hole profile.

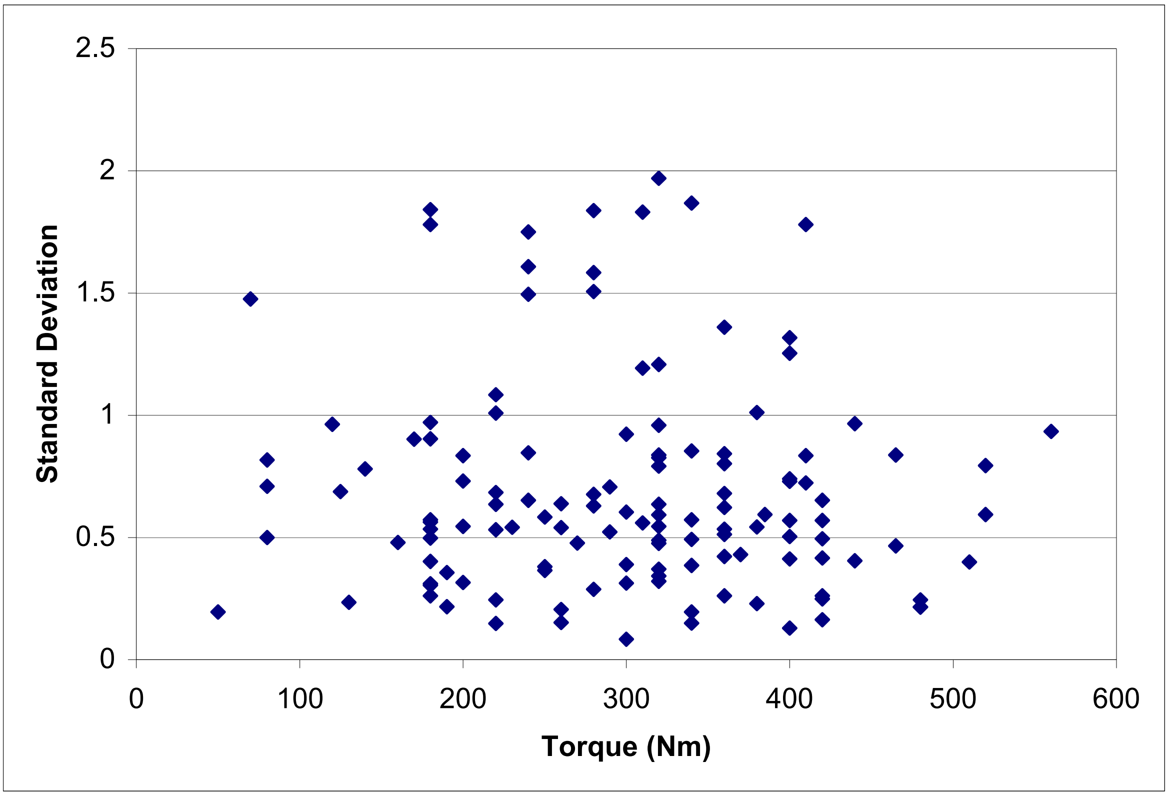


***Figure 6-20  Torque against hole profile standard deviation***

Similarly, Figure 6-21 shows no correlation between the standard deviation and thrust.

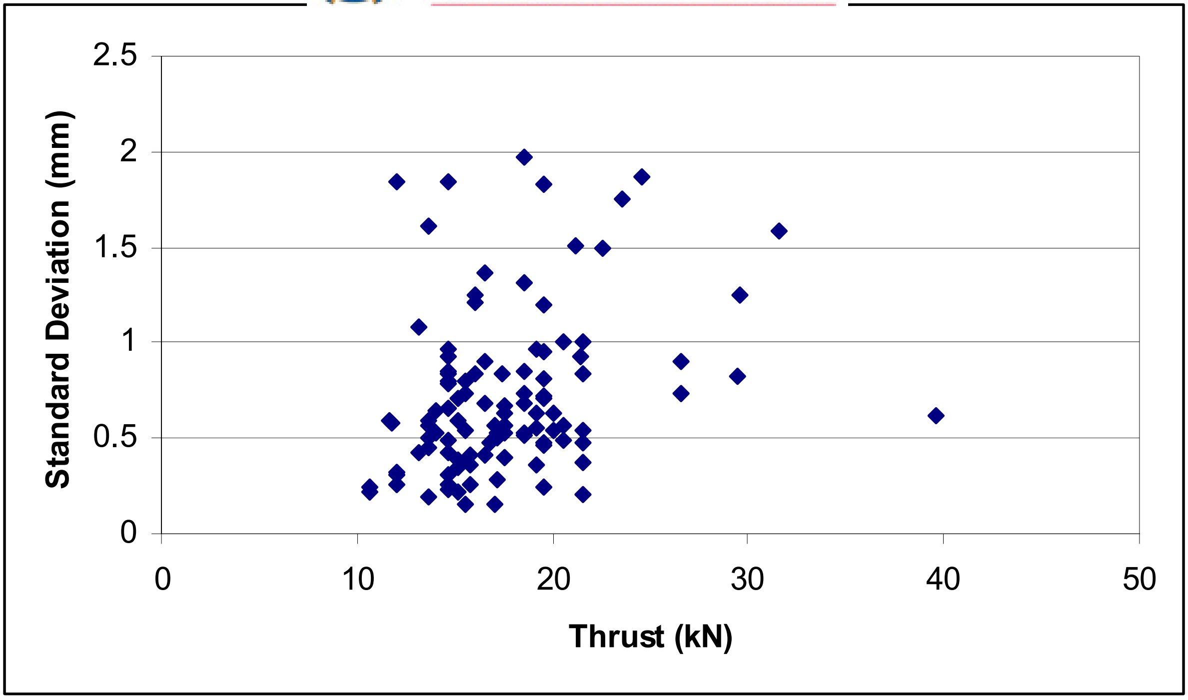


***Figure 6-21 Thrust against hole profile standard deviation***

Figure 6-22 shows the relationship between drilling speed and hole quality for wet flushing systems only. Again, no correlation is evident. A similar analysis is also conducted for dry drilling machines (Figure 6-23), again showing no obvious correlation.

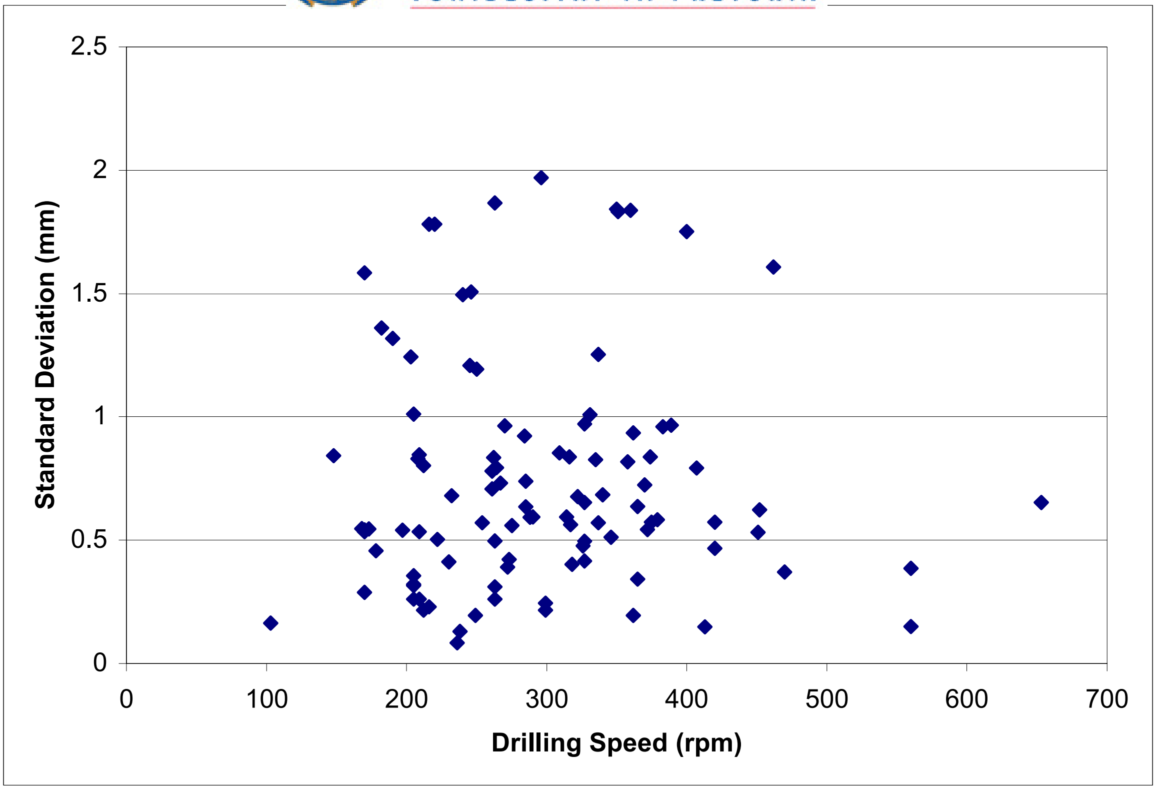


***Figure 6-22 Drilling Speed against hole profile standard deviation in machines using wet flushing system***

While the comparison between wet drilling machines and dry drilling machines must be made, Figure 6-22 and Figure 6-23 illustrate that dry drilling machines, on average, drill at higher speeds than their wet counterparts, rather than produce any discernable difference in hole quality.

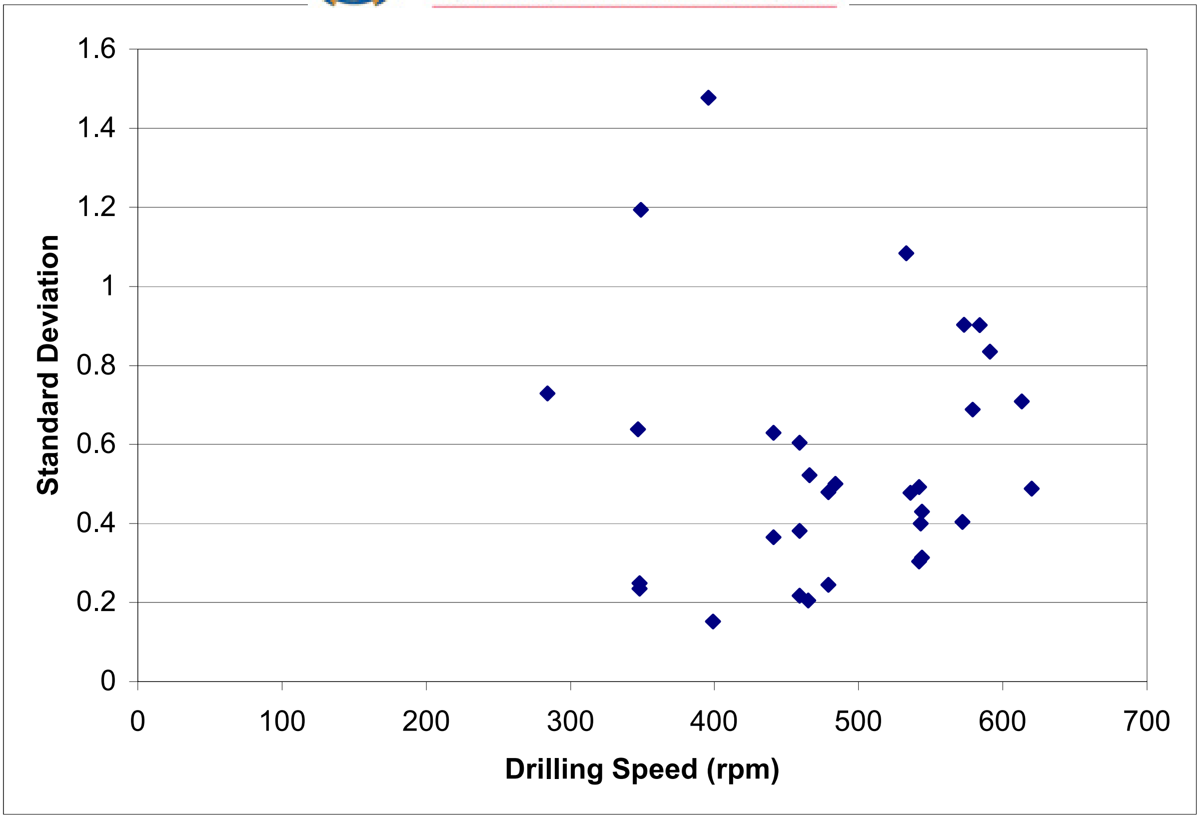


***Figure 6-23 Drilling speed against hole profile standard deviation in machines using dry flushing system***

The relationship between torque and hole quality for dry drilling machines is presented in Figure 6-24. Similarly, no correlation is evident.

The final parameter that was checked against hole profile was resin spinning speed Figure 6-25. It was also found that there is no correlation between the hole profile and resin spinning speed.

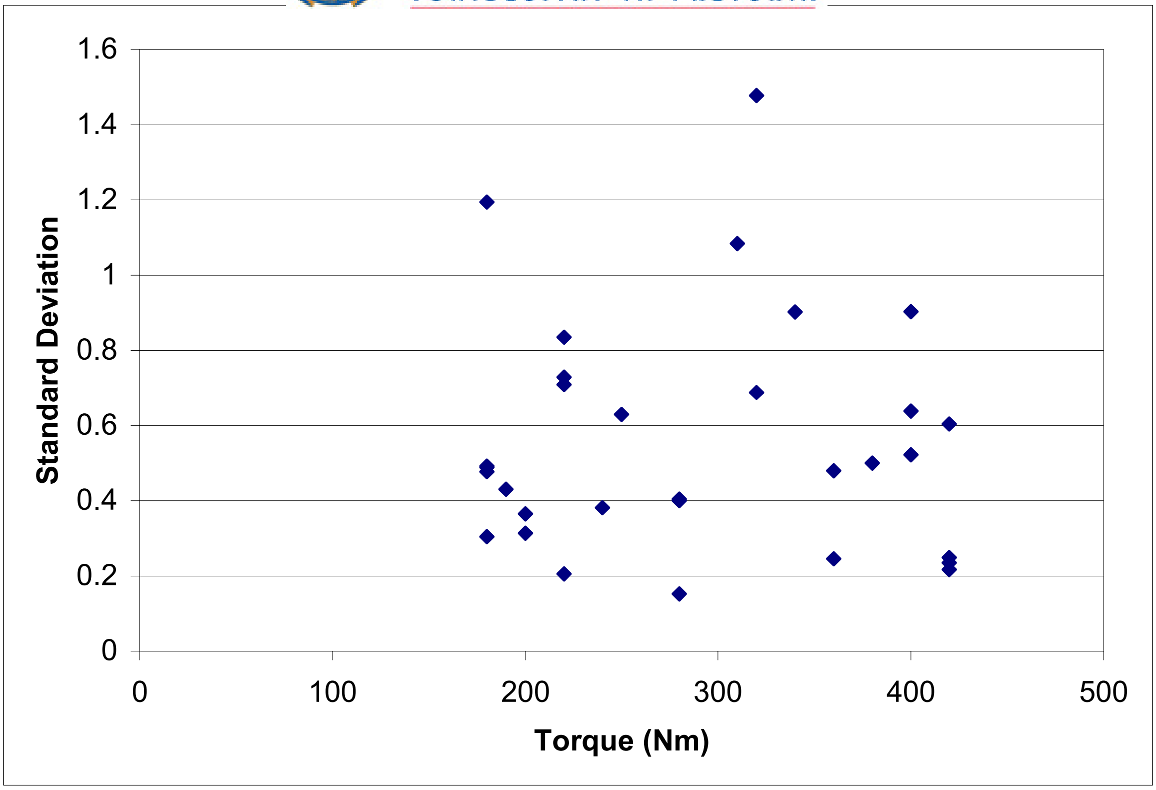


***Figure 6-24 Torque against hole profile standard deviation in machines using dry flushing system***

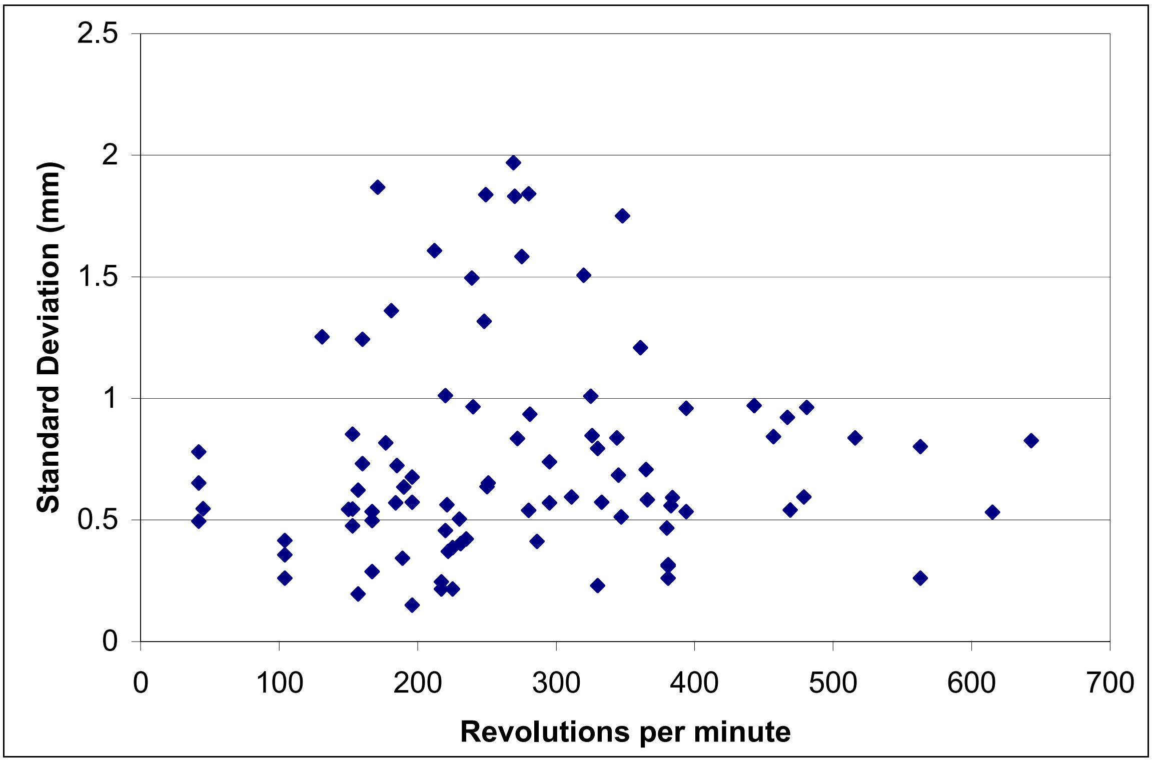


***Figure 6-25 Resin spinning speed against hole profile standard deviation in machines using wet flushing system***

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Figure 6-26 and Figure 6-27 show that there is very little difference between standard deviation of hole profiles in sandstone and in the softer materials such as siltstone, shale or coal. While there is more variation in the case of sandstone, in both cases the mean standard deviation is approximately 0.6 mm.

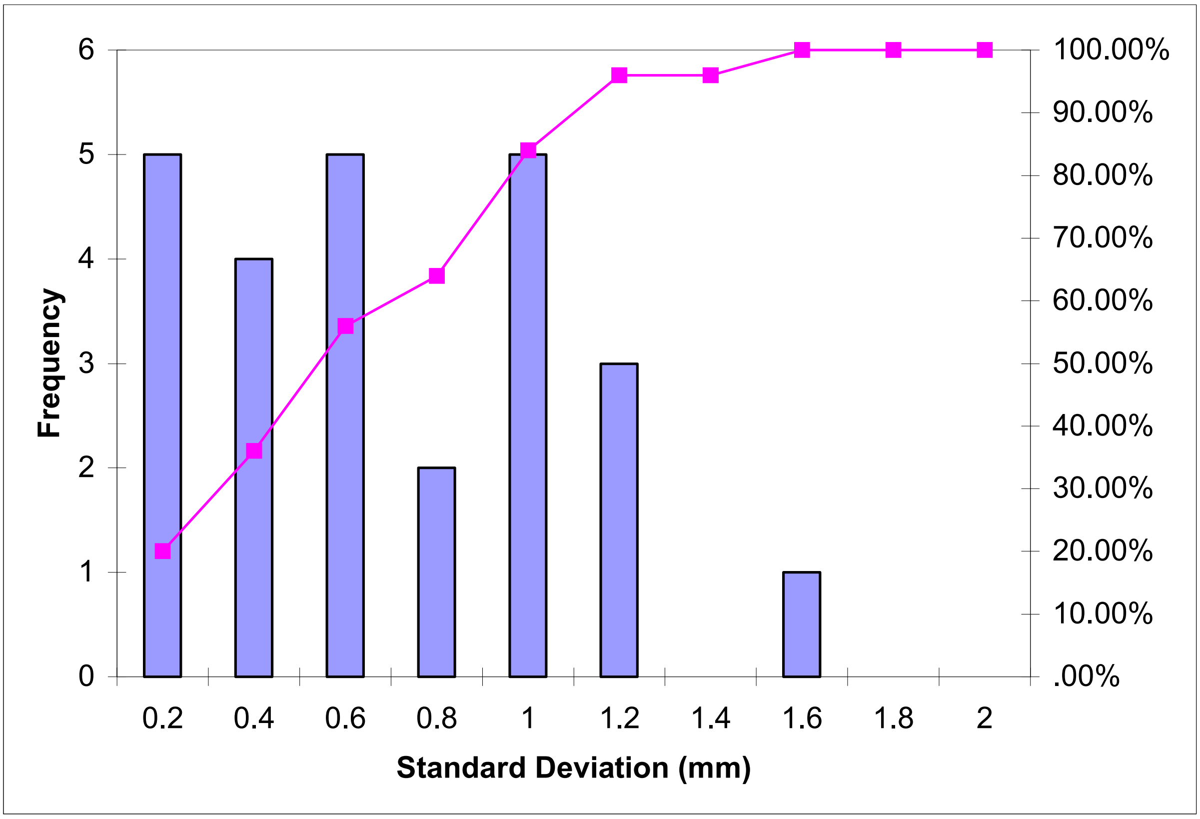


***Figure 6-26 Hole profile standard deviation in sandstone***

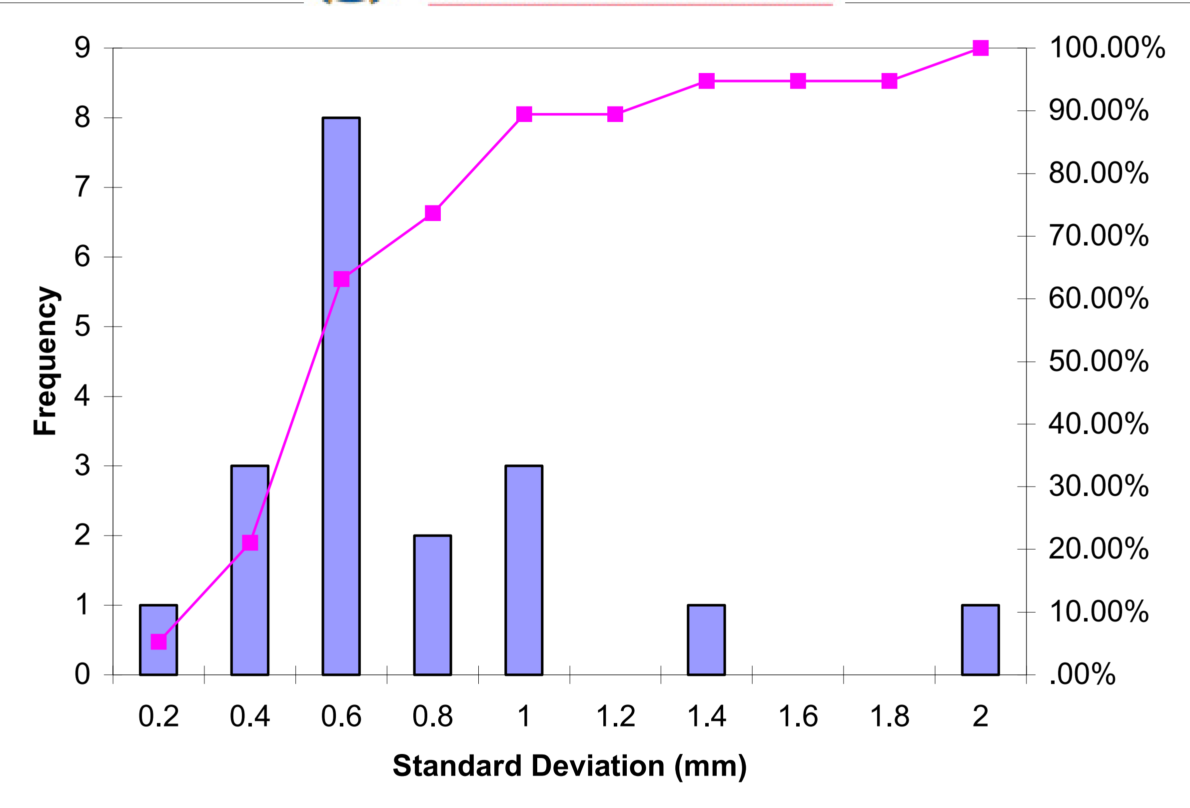


***Figure 6-27 Hole profile standard deviation in 'soft' materials***

### 6.2.4 Effect of wet and dry drilling

The effect of wet and dry drilling is one of the most discussed topics of roof bolting. However, there are not many scientific investigations relating to this effect. Therefore, a total of 24 short encapsulated pull tests (SEPT) using the standard testing procedure of the ISRM (ISRM, 1985) were conducted to determine the effect of wet and dry drilling. These tests were conducted for three different resin types; namely, 15-second resin, 30-second resin and 5/10-minute resin using the same roofbolter, and the same resin from Manufacturer “B”.

Figure 6-28 shows the bond strengths achieved for different resin types using wet and dry drilling. This figure indicates that bond strengths for wet drilling are between 4 to 28 per cent greater than with dry drilling probably due to the fine particles which may be left behind after dry drilling.

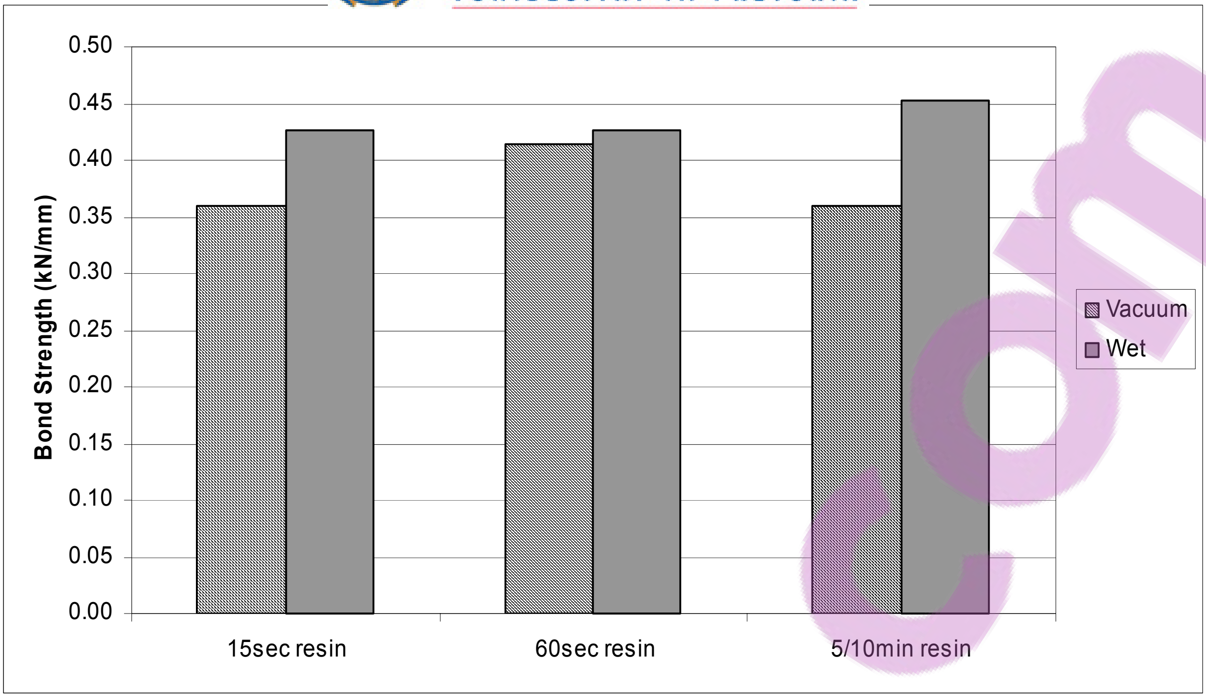


***Figure 6-28 Effect of wet-dry drilling***

Figure 6-29 shows the overall stiffnesses achieved (maximum load achieved / displacement at maximum load) when wet and dry drilling is used for different resins. As can be seen from this figure, the overall stiffnesses are significantly greater for wet drilling than for dry drilling for the faster speed resin types.

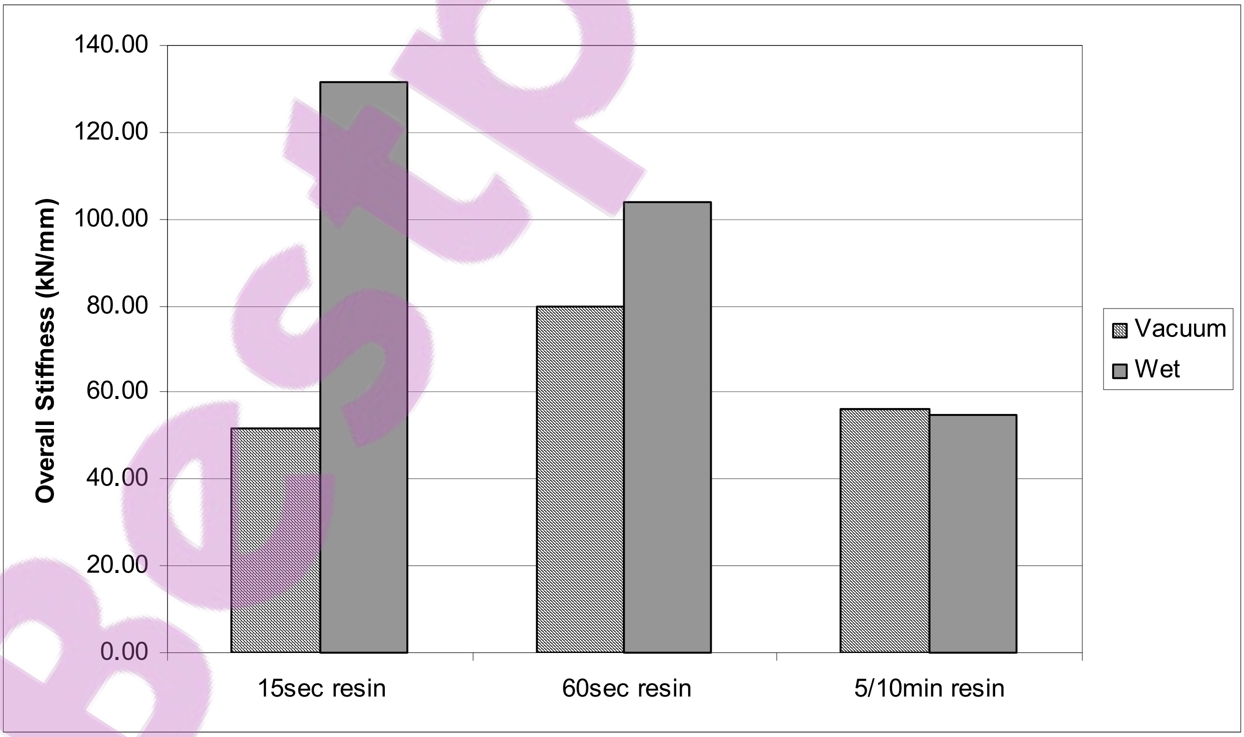


***Figure 6-29 Effect of wet and dry drilling on overall support stiffness***

The data shown in the above figures is presented in Table 6-1.

***Table 6-1 Effect of wet and dry drilling (averages)***

| Rock Type | Drill type | Resin Type | Annulus (mm) | Bond Strength (kN/mm) | Contact Shear Strength (kPa) | Max Load Achieved (kN) | Overall Stiffness (kN/mm) |
|---|---|---|---|---|---|---|---|
| Shale | Vacuum | 15-second | 4.22 | 0.36 | 4029.22 | 90.00 | 51.72 |
| Shale | Wet | 15-second | 3.93 | 0.43 | 4908.03 | 106.67 | 131.71 |
| Shale | Vacuum | 60-second | 4.30 | 0.41 | 4632.19 | 103.33 | 79.88 |
| Shale | Wet | 60-second | 3.63 | 0.43 | 4974.21 | 106.67 | 103.77 |
| Shale | Vacuum | 5/10-minute | 4.55 | 0.36 | 3964.22 | 90.00 | 56.08 |
| Shale | Wet | 5/10-minute | 3.35 | 0.45 | 5404.71 | 113.33 | 55.04 |

## 6.3 Performance of roof bolts

### 6.3.1 Performance of roof bolts manufactured in South Africa

A total of 61 short encapsulated pull tests using the standard ISRM testing procedure (ISRM, 1985) were conducted on 20 mm roof bolts to determine the performance of bolts obtained from four manufacturers.

The results from these tests are shown in Figure 6-30. As can be seen from this figure, bolts from all four manufacturers showed almost identical results in sandstone, while in shale the results were dissimilar. This figure also indicates that bolts from Manufacturer "A" performed relatively better in shale compare to Manufacturer "B".

As will be shown in the following chapters, the roof bolt profile plays a significant role in determining the pull-out resistance of roof bolts. However, Figure 6-30 indicates that the variation in the performance of roof bolts in sandstone is not significant. In shale, however, there appears to be a significant variation in pull-out strength. This variation can directly be attributed to the profiles of different roof bolts (see Section 6.3.3).

The data shown in the above figure is presented in Table 6-2.

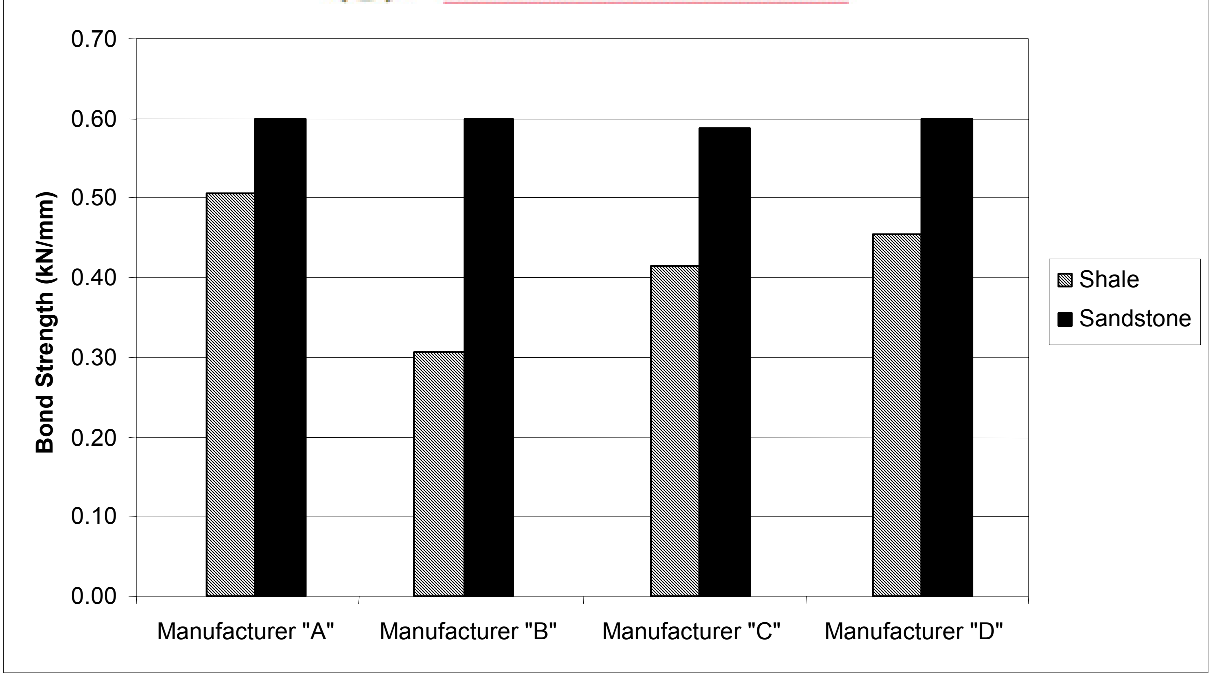


***Figure 6-30 Performance of roof bolts determined from underground SEPTs***

***Table 6-2 Performance of roof bolts determined from underground SEPTs (averages)***

| Rock Type | Manufacturer | Hole Annulus (mm) | Bond Strength (kN/mm) | Contact Shear Strength (kPa) | Max Load Achieved (kN) | Overall Stiffness (kN/mm) |
|---|---|---|---|---|---|---|
| Shale | A | 3.30 | 0.51 | 6036.35 | 126.67 | 101.94 |
| Shale | B | 4.45 | 0.31 | 3406.12 | 76.67 | 81.23 |
| Shale | C | 3.35 | 0.41 | 4920.62 | 103.33 | 40.26 |
| Shale | D | 3.67 | 0.45 | 5318.22 | 113.33 | 23.82 |
| Sandstone | A | 2.96 | 0.60 | 7330.47 | 150.00 | 128.48 |
| Sandstone | B | 3.02 | 0.60 | 7281.30 | 150.00 | 208.77 |
| Sandstone | C | 3.49 | 0.59 | 6926.54 | 146.67 | 30.88 |
| Sandstone | D | 3.50 | 0.60 | 7045.31 | 150.00 | 69.56 |

### 6.3.2 Tensioned versus non-tensioned roof bolts

An additional 25 short encapsulated pull tests were conducted to determine the effect of tensioning on bond strength. These tests were conducted in sandstone and shale roofs.

Figure 6-31 shows the effect of tensioning on bond strength. Non-tensioned roof bolts achieved significantly greater bond strengths then the tensioned bolts. Figure 6-32 shows the effect of

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tensioning on overall support stiffness. Similarly, non-tensioned roof bolts achieved significantly stiffer systems than the tensioned roof bolts.

Although this finding may be significant from the spin-to-stall support system point of view, it is thought that with tensioned bolts, because the bond length is only 250 mm, the bonding could easily be damaged when the bolt is being tensioned. For this reason it is probable that the test results obtained do not give a fair reflection of the performance of tensioned bolts. It is therefore suggested that a new testing procedure be developed to test the performance of tensioned bolts.

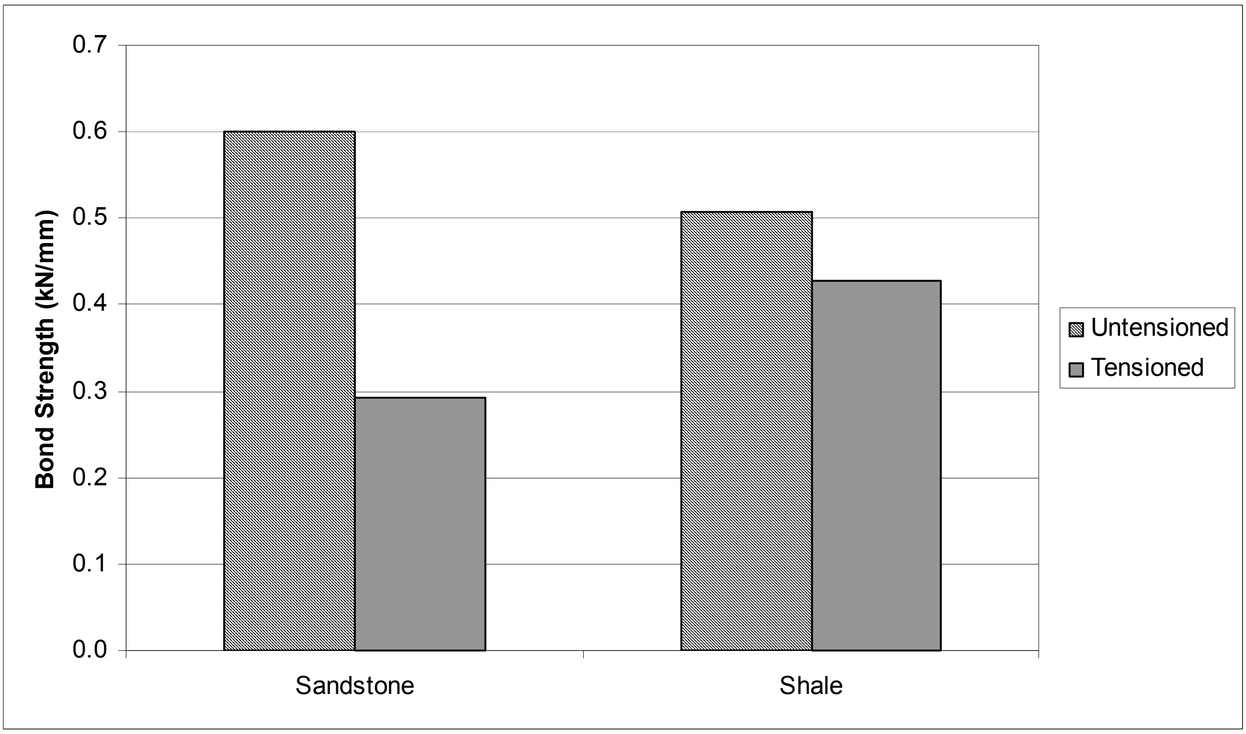


***Figure 6-31 Effect of tensioning on bond strength***

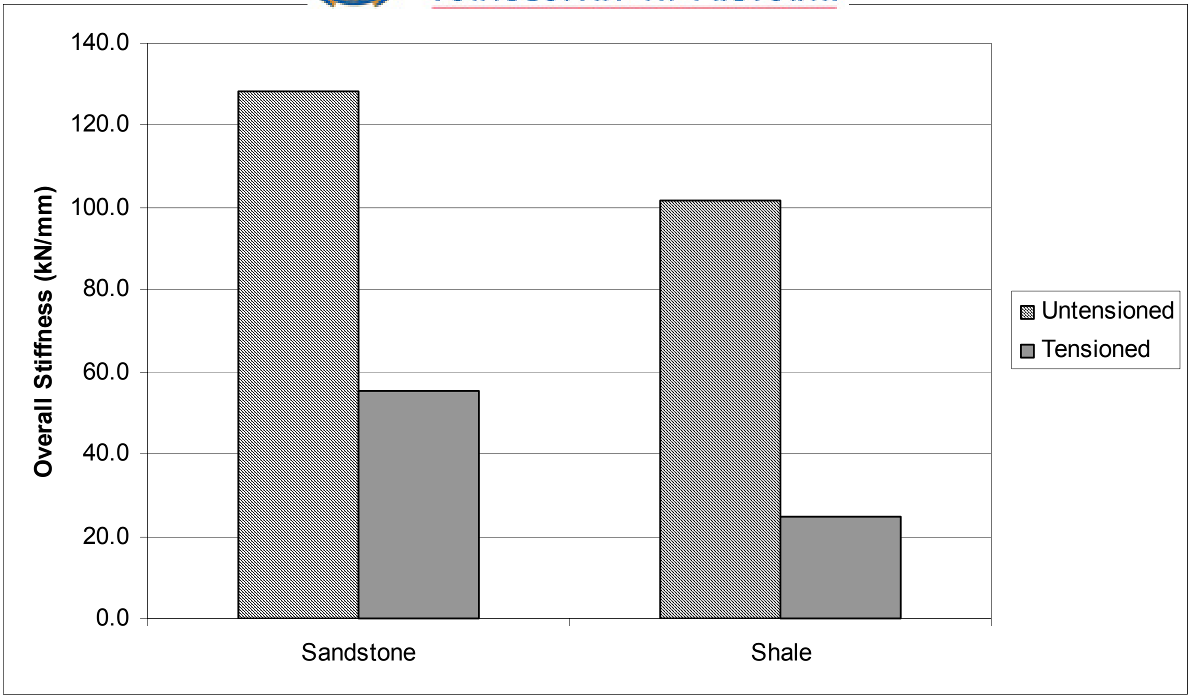


***Figure 6-32 Effect of tensioning on overall stiffness***

The data shown in the above figures is presented in Table 6-3.

***Table 6-3 Effect of tensioning on support performance (averages)***

| Rock Type | Type | Annulus (mm) | Bond Strength (kN/mm) | Contact Shear Strength (kPa) | Max Load Achieved (kN) | Overall Stiffness (kN/mm) |
|---|---|---|---|---|---|---|
| Sandstone | Non-tensioned | 2.96 | 0.60 | 7330.47 | 150.00 | 128.48 |
| Sandstone | Tensioned | 3.87 | 0.29 | 3375.81 | 73.33 | 55.25 |
| Shale | Non-tensioned | 3.30 | 0.51 | 6036.35 | 126.67 | 101.94 |
| Shale | Tensioned 5 | 3.35 | 0.43 | 5131.66 | 106.67 | 24.54 |

### 6.3.3 Variation in roof bolt parameters

In a support system, it may not be possible to control the hole diameter, because of many factors, such as the rock strength, bit type, drilling type, thrust of roofbolter etc. However, it is possible to control the bolt diameter and profile, which is a part of the engineering design. Therefore, an investigation into the variations in the roof bolts that are currently being used in South Africa was conducted by means of measuring the bolt core diameters and rib diameters from different bolt manufacturers in South Africa.

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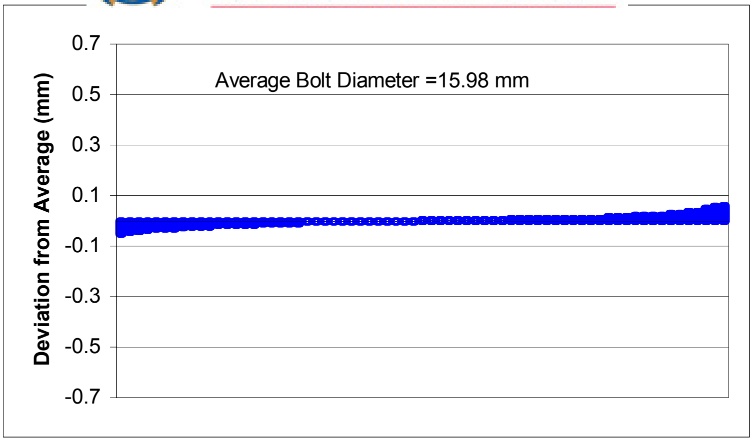
A total of 235 roof bolts from three different manufacturers were evaluated (approximately 80 roof bolts from each manufacturer). The bolts were measured in three places - top, middle and above the thread - to give an average bolt diameter. Rib diameter was measured diagonally across both ribs and bolt core diameter was measured between the ridges, normal to the axis of the bolt.

Bolts of 16 mm diameter were measured from Manufacturers “A” and “B”, and 20 mm roof bolts were measured from Manufacturer “C”. Manufacturer “D” did not supply roof bolts for testing as part of this task of the project. Therefore, they are excluded from this investigation.

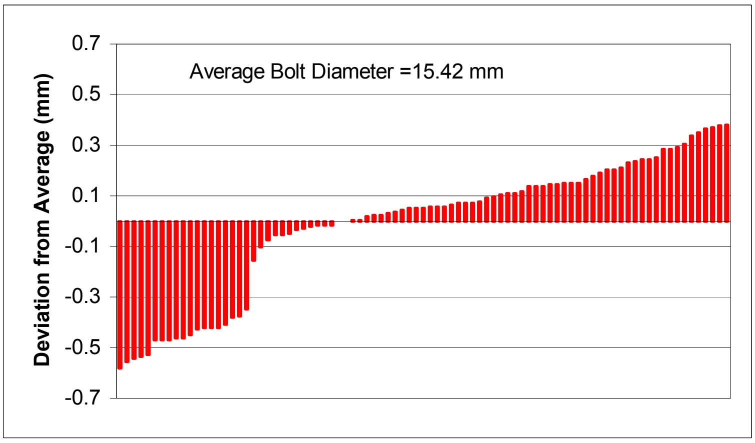
Figure 6-33 shows the deviations of roof bolt diameters (from the average) and the average roof bolt diameters from these three manufacturers. This figure highlights that the deviations from the average diameters of roof bolts from Manufacturers “A” and “C” will be in a significantly narrower range than those from Manufacturer “B”. As shown in Figure 6-30, the bolts from Manufacturer “A” performed relatively better than bolts from Manufacturer “B” in shale rock type.

The rib diameter measurements from these three manufacturers are presented in Figure 6-34. This Figure shows that there is a significant variation in the rib-heights of the roof bolts from Manufacturer “B” and that the average rib-height of roof bolts from this manufacturer is approximately 34 per cent less than those supplied by the other two manufacturers.

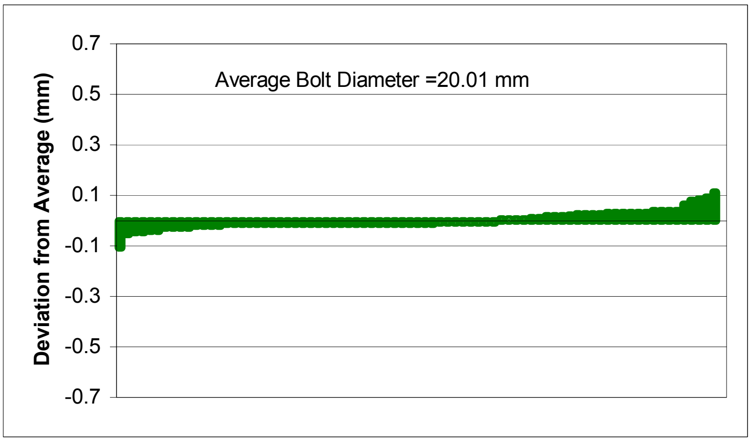
The effect of annulus size on support performance has been shown to be significant. Also, theoretically, a 0.6 mm reduction in bolt diameter can reduce the yield load of a 16 mm bolt by 7 per cent (assuming a tensile strength of 480 MPa). This highlights the need for quality control procedures to be in place at mines for checking the elements of a support system, which are themselves part of the engineering design (roof bolt, bits etc.).




*Manufacturer "A"*

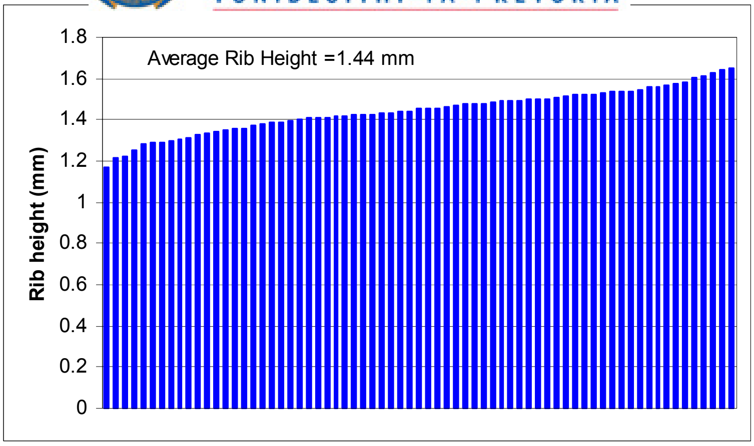



*Manufacturer "B"*

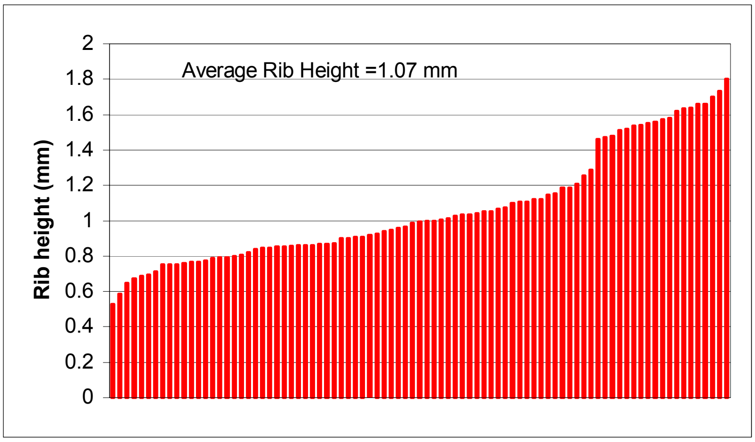



*Manufacturer "C"*

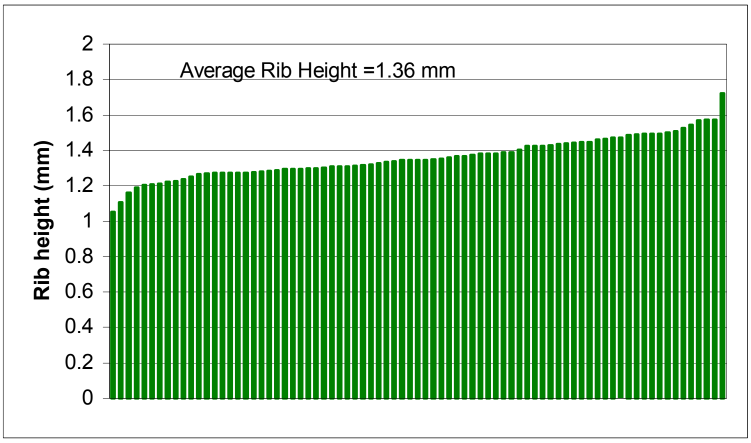
***Figure 6-33 Roof bolt diameter deviations in bolts from three different manufacturers***




*Manufacturer "A"*




*Manufacturer "B"*




*Manufacturer "C"*

***Figure 6-34 Roof bolt rib-height measurements in bolts from three different manufacturers***

An attempt was also made to determine the rib thickness, the spacing between the ribs, and the angle of the ribs of currently used roof bolts in South Africa. Approximately 30 roof bolts from four different suppliers were obtained and three measurements were taken for each bolt. The average results obtained from each manufacturer are shown in Table 6-4.

*Table 6-4 Rib thickness, spacing and angle measured on South African roof bolts*

| Bolt Manufacturer | Rib thickness (mm) | Spacing between the ribs (mm) | Rib angle (degree) |
|---|---|---|---|
| "A" | 3.88 | 8.70 | 64 |
| "B" | 3.02 | 7.33 | 70 |
| "C" | 3.47 | 10.79 | 63 |
| "D" | 3.04 | 9.40 | 60 |
| Average | 3.35 | 9.06 | 64.25 |

As can be seen from this table, there are differences between the parameters that determine the bolt profile in South African roof bolts. Figure 6-35 shows the bolts from the four different manufacturers. However, the influence of these small differences on bolt performance is difficult to determine. It is therefore recommended that a laboratory testing programme be carried out to determine the effect of these parameters on the performance of roof bolts being used in South Africa and to optimise the design.

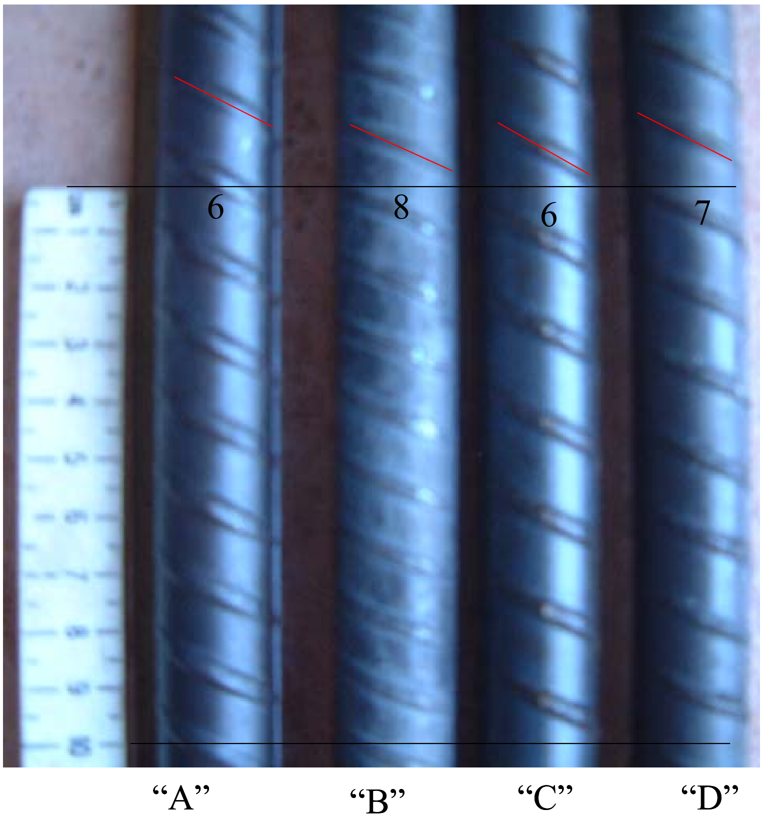



*Figure 6-35 Visual illustration of four South African roof bolts*

Although there are small differences between the South African roof bolts tested, there is a significant visual difference between the AT bolt from the UK and typical South African bolts (Figure 6-36). The angle of ribs between the two types of bolt is significantly different. A detailed

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sensitivity analysis to the various parameters should be conducted on the resin that would be used and the rock types in which it would be installed in South African collieries.

Roof bolting should be considered as a system and the design of elements comprising the system should be such that the difference in strength between the weakest and strongest element is minimised.


***Figure 6-36    Visual comparison of UK and South African bolts***

## 6.4 Performance of resin

A total of 132 short encapsulated pull tests using the standard ISRM testing procedure (ISRM, 1985) were conducted to determine the performance of various resin types obtained from two manufacturers, namely Manufacturer "A" and Manufacturer "B".

The results from these tests in three different rock types are shown in Figure 6-37, Figure 6-38 and Figure 6-39. These figures indicate that, in sandstone, 15 second and 30 second resin types from the two different manufacturers performed similarly. However, the performance of slow 5/10-minute resins from both manufacturers was much lower than that of the fast resins. In all short encapsulated pull tests, the bolts were pulled 24 hours after the installation. The large discrepancy between bond strengths for the 5/10-minute resins may be entirely due to not enough waiting time. This finding contradicts with findings of van der Merwe (1989) and therefore should be investigated in detail to determine the effect of slow setting resin on overall system performance by overcoring the full-column resin bolts underground.

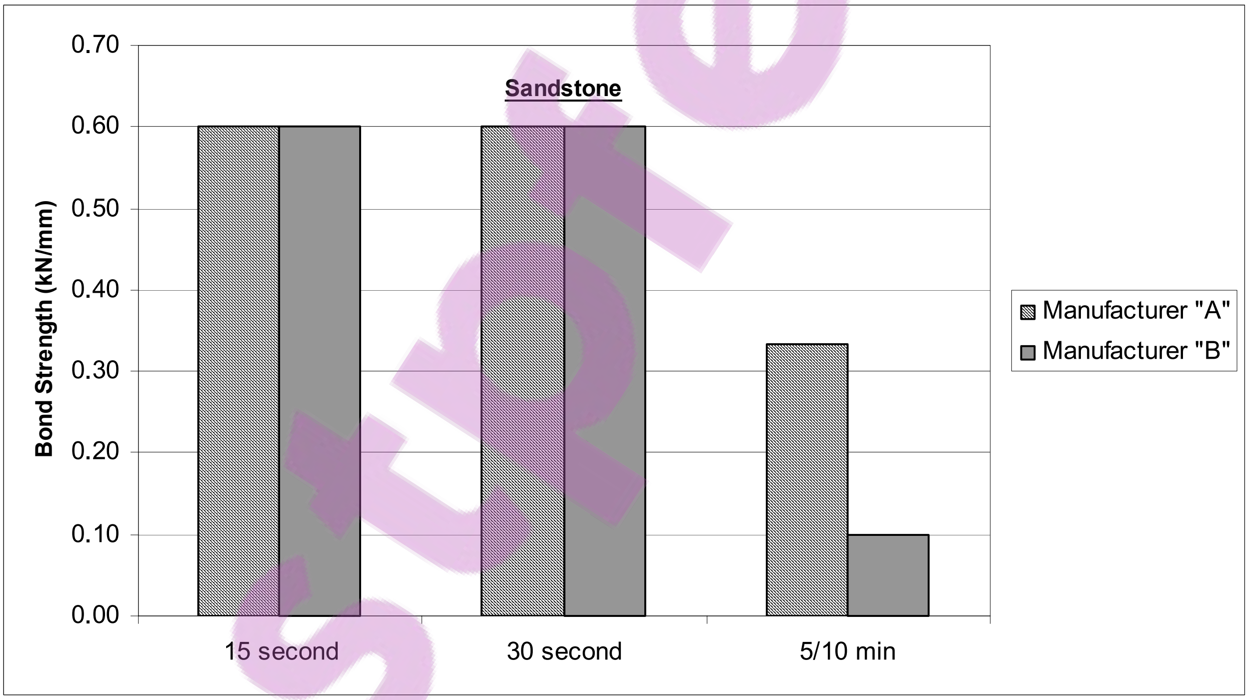


***Figure 6-37 Performance of 15-second and 30-second resin types in sandstone from both resin manufacturers***

No trend could be observed in comparing the resin performance in coal and shale.

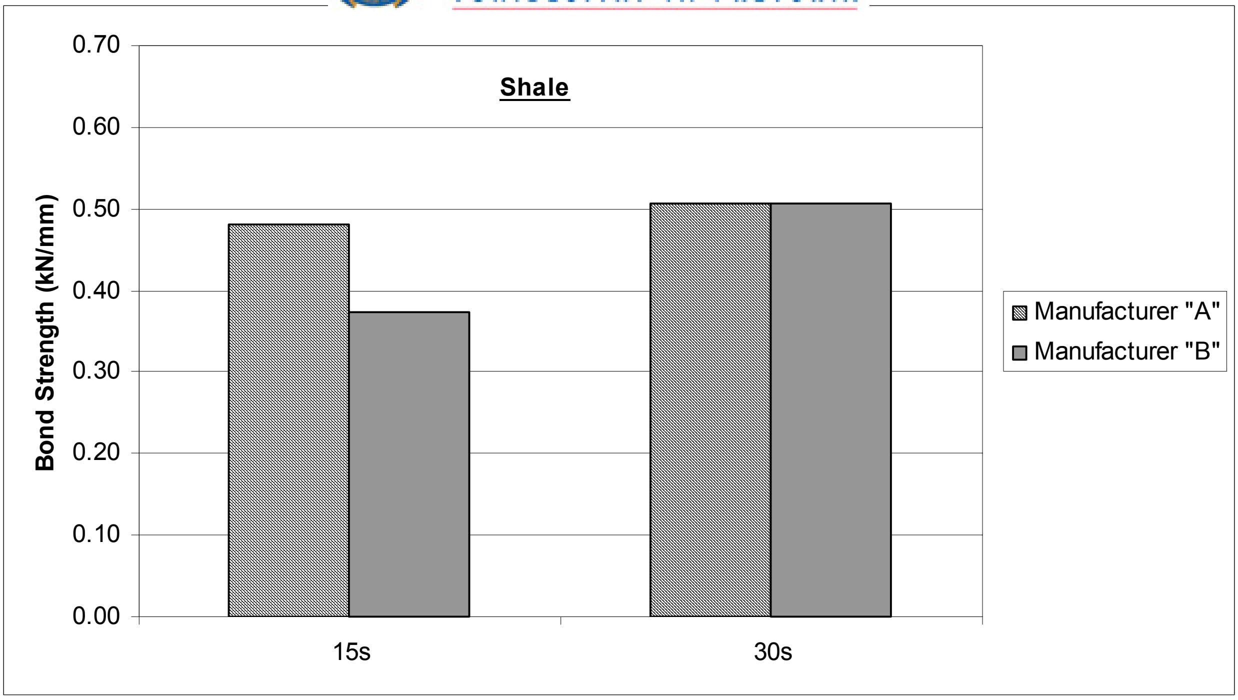


***Figure 6-38 Performance of 15-second and 30-second resin types in shale from both resin manufacturers***

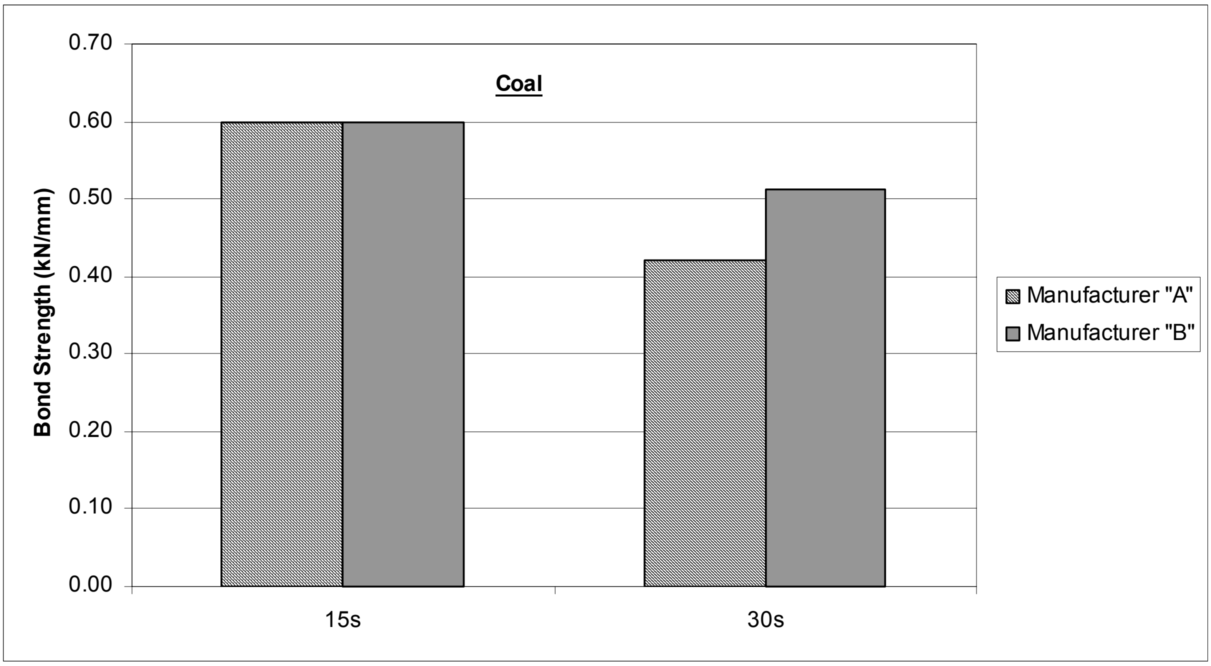


***Figure 6-39 Performance of 15-second and 30-second resin types in coal from both resin manufacturers***

An analysis of the system stiffness of both resin types from both manufacturers was also conducted. The results are shown in Figure 6-40.

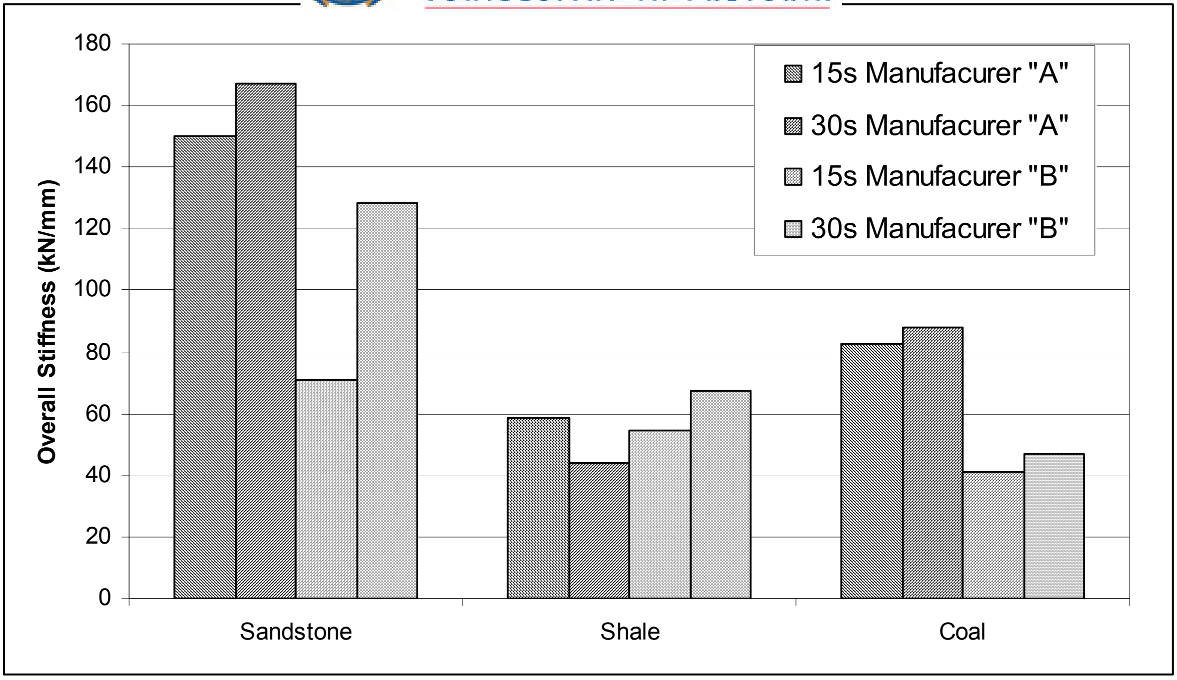


***Figure 6-40 System stiffness of 15-second and 30-second resin types from both resin manufacturers***

Figure 6.4 indicates that both 15-second and 30-second resins from Manufacturer “A” achieved higher stiffness than those from Manufacturer “B” in sandstone and coal. In shale, both resins from both manufacturers performed in a similar manner.

The data shown in above figures is presented in Table 6-5.

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***Table 6-5 Overall stiffnesses of resin determined from underground SEPTs (averages)***

| Rock Type | Manufacturer | Resin Type | Annulus (mm) | Bond Strength (kN/mm) | Contact Shear Strength (kPa) | Max Load Achieved (kN) | Overall Stiffness (kN/mm) |
|---|---|---|---|---|---|---|---|
| Sandstone | A | 15-second | 3.37 | 0.60 | 7170.96 | 150.00 | 150.35 |
| Sandstone | A | 30-second | 3.80 | 0.60 | 6980.67 | 150.00 | 167.35 |
| Sandstone | A | 5/10-minute | 3.17 | 0.33 | 4013.31 | 83.33 | 65.56 |
| Sandstone | B | 15-second | 3.01 | 0.60 | 7299.21 | 150.00 | 71.23 |
| Sandstone | B | 30-second | 2.96 | 0.60 | 7330.47 | 150.00 | 128.48 |
| Sandstone | B | 5/10-minute | 3.33 | 0.11 | 1184.60 | 25.00 | 22.03 |
| Shale | A | 15-second | 3.45 | 0.48 | 5689.04 | 120.00 | 58.53 |
| Shale | A | 30-second | 3.37 | 0.51 | 6034.17 | 126.67 | 43.88 |
| Shale | B | 120-second | 3.65 | 0.39 | 4613.01 | 98.33 | 24.51 |
| Shale | B | 15-second | 3.22 | 0.37 | 4497.89 | 93.33 | 54.33 |
| Shale | B | 30-second | 3.30 | 0.51 | 6036.35 | 126.67 | 67.66 |
| Shale | B | 5/10-minute | 3.27 | 0.49 | 5957.16 | 123.33 | 42.99 |
| Coal | A | 15-second | 3.55 | 0.60 | 7056.66 | 150.00 | 82.46 |
| Coal | A | 30-second | 3.43 | 0.42 | 4901.13 | 105.00 | 88.10 |
| Coal | B | 15-second | 3.48 | 0.60 | 7100.73 | 150.00 | 40.86 |
| Coal | B | 30-second | 3.50 | 0.51 | 5963.47 | 128.33 | 47.19 |

## 6.5 Specifications for bolt and resin

The deform pattern of a bolt is an important factor in determining the support system performance. The bolt profile determines three important phases of support installation and performance. These are:

- Quality of resin mixing;
- Pushing the resin towards the end of the hole; and
- Load transfer capabilities of the bolting system.

However, the effect of bolt profile on support performance is poorly understood by the end user. The majority of information pertaining to the design and specification of fully encapsulated roof bolting systems is commercial intellectual property, and little information is available in the public domain. One of the causes of this lack of knowledge regarding the influence of bolt profile on support performance is the testing procedure adopted. When testing the effect of bolt profile, the important factor is the location of the failure mechanism, which should be on the resin-bolt

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interface. Extensive laboratory short encapsulated pull tests resulted in inconsistent results due to failure taking place on the rock- or pipe-resin interface. In this case, the maximum load in the test is probably independent of bolt profile, assuming that bolt profile did not affect the quality of resin mixing.

The important considerations in a roof bolt profile are depicted in Figure 6-41:

- The rib radius ($R$);
- Rib angle ($\alpha$);
- Distance between the ribs ($p$); and
- Thickness of rib ($d$).

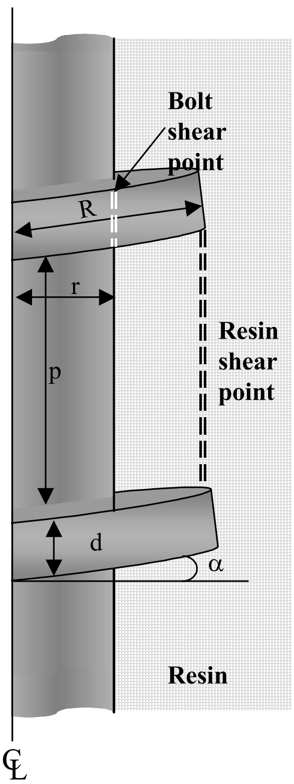


***Figure 6-41 Simplified drawing of roof bolt profile components***

Matching the bolt profile to resin strength is also an important consideration in support system design. In 1999, the South African coal mining industry imported Australian low-rib height roof bolts, which showed relatively poor performance (O'Connor, 2004).

O'Connor (2004) developed a mathematical model to determine the effectiveness of matching resin properties to the profile of the bolt. This model is based on the bolt shearing at the base of the ribs, at the same load as the grout shears between the ribs. O'Connor stated that this happens when:

$$\frac{\text{Resin shear strength}}{\text{Steel shear strength}} = \frac{d\,r}{R\,p} \qquad [6\text{-}1]$$

Where $R$ is the rib radius, $\alpha$ is the rib angle, $p$ is distance between the ribs, and $d$ is the thickness of rib.

This equation indicates that to maintain a balanced performance between resin and roof bolt profile, lower resin strength requires either higher ribs, or longer spacing between ribs, or both of these. Note that this model ignores the effects of resin mixing, film shredding and rib angle.

This model also indicates that the maximum pull-out loads can be achieved between the resin and roof bolt when:

- The ribs are relatively high;
- The distance between the ribs is relatively low; and
- The ribs are relatively thick.

It should also be noted that the failure between the rock and the resin takes place in a similar manner in a short encapsulated pull test. Therefore, the pull-out loads (from SEPT) in stronger rock (such as sandstone) are greater than in softer rock, such as shale (Figure 6-42) due to the nature of greater shear strength of sandstone.

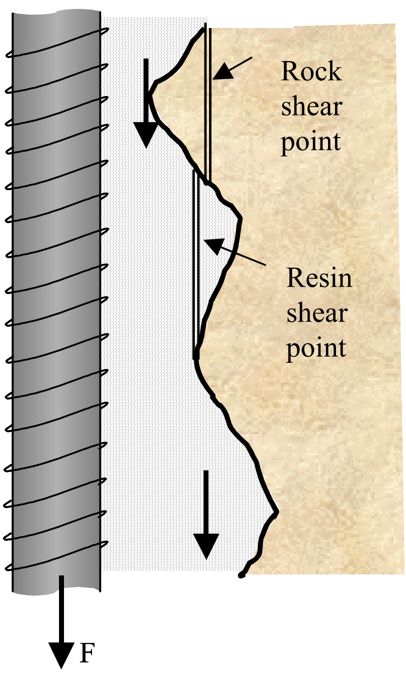


***Figure 6-42 Simplified drawing of failure between the rock and the resin***

As can be seen from Figure 6-42 and Equation [6-1], the pull-out load to failure will increase:

- When the rock shear strength is relatively high; and
- When the hole is rougher.

From all of the above it can be concluded that the failure characteristics of a roof bolting system will be determined by the shear strength of bolt / resin / rock interface:

- The failure will take place at the resin-rock interface when the shear strength of the rock is lower than the resin (rock will fail);
- The failure will take place at either the resin-rock or resin-bolt interface when the resin shear strength is the lowest in the system;
- When the resin shear strength is the lowest in the system, the failure will be determined by the roughness of the hole and the bolt profile.

The other important consideration in the performance of a roof bolt is the bolt geometry (Figure 6-43). The effect of rib angle on the pull-out resistance can be calculated with the use of the following formula:

$$F_R = F\, Cos\alpha \qquad [6\text{-}2]$$

Where $F_R$ is reaction force, $F$ is applied pull-out load and $\alpha$ is rib angle.

Equation [6-2] shows that as the rib angle increases the pull-out load of a bolt decreases. It is therefore suggested that in order for relatively high pull-out loads to be achieved, low rib angles are required. This requirement was confirmed by laboratory tests on different bolts with different rib angles in Australia (O'Brien, 2003). However, lowering the rib angle may result in poor resin mixing performance. It is therefore recommended that further work on the effect of bolt geometry on roof bolt performance be carried out. Such work will then allow the performance of roof bolts to be determined by engineering design that could differ for different rock types. Bolt design could be optimised with the aim of inducing failure on this interface. It is also recommended that the quality of resin mixing should be investigated with different rib angles for determining the most effective rib angles on the roof bolts. Unfortunately, the very similar rib combinations in South African bolt types and testing in an underground environment (uncontrolled conditions) meant that the effect of rib angle, rib height and thicknesses and spacing between the ribs could not be quantified. It is therefore suggested that these tests should be conducted in a controlled laboratory environment.

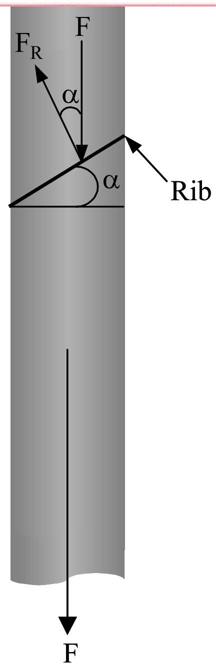


***Figure 6-43  Effect of rib angle on pull-out loads (simplified)***

## 6.6 Effect of bit, annulus and rock type

### 6.6.1 Performance of bits

Two types of drill bits are commonly used in South African collieries. These are the 2-prong bits and the spade bit. Both bits are shown in Figure 6-44.


***Figure 6-44  Spade and 2-prong bits (25 mm)***

A total of 40 short encapsulated pull tests were conducted in order that the performance of the two different bit types could be determined.

The results from these tests in sandstone and shale are summarised in Figure 6-45. As can be seen in the figure, the 2-prong bit outperformed the spade bit in both rock types. However, the annuli obtained from the 2-prong bit were always greater than those from the spade bit (Figure 6-46). This is probably because of rougher holes obtained with 2-prong bits.

The stiffnesses obtained from the 2-prong bits were also greater than those from the spade bit (Figure 6-47). These findings suggest that 2-prong bits are more effective in collieries than the spade bits.

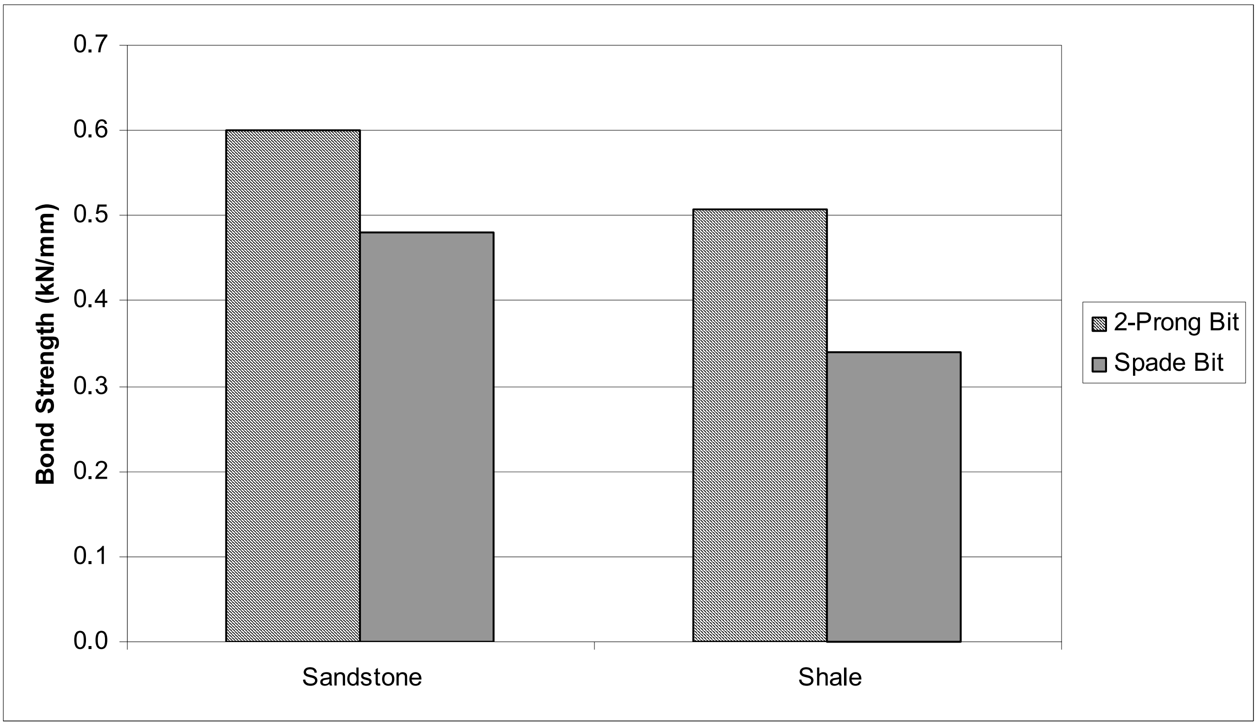


***Figure 6-45 Performance of spade bit and 2-prong bit***

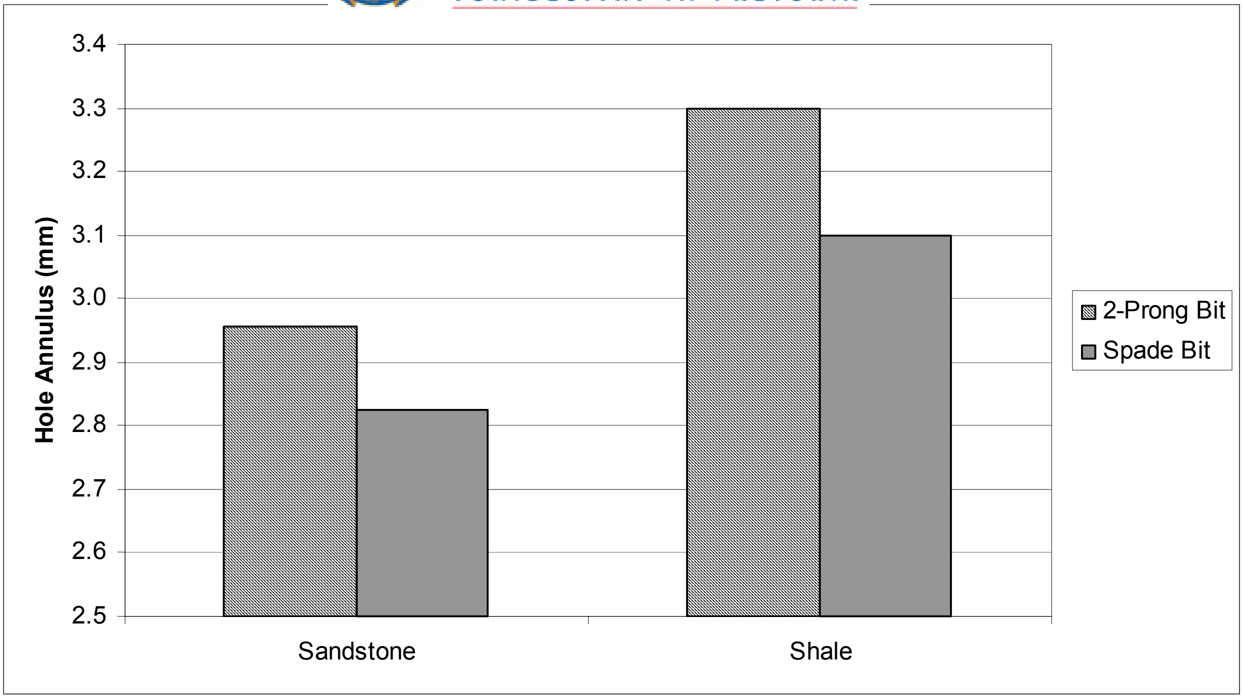


***Figure 6-46 Hole annuli obtained from the 2-prong and spade bits***

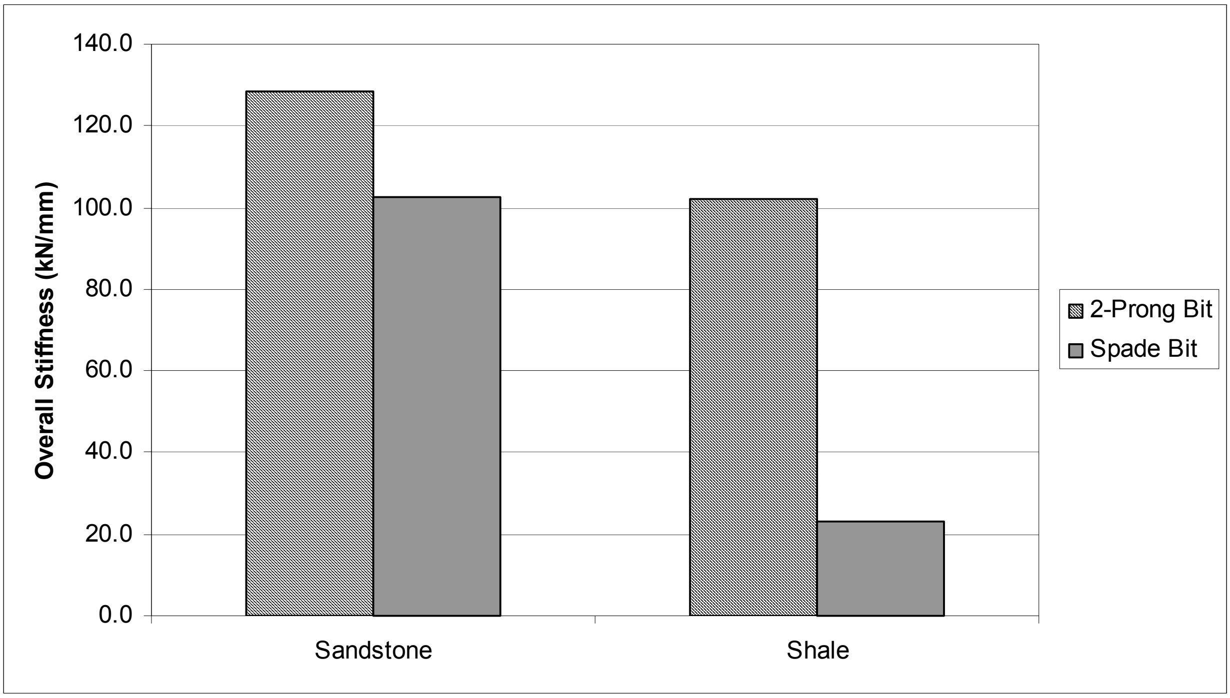


***Figure 6-47 Overall stiffnesses obtained from the 2-prong and spade bits***

The data shown in the above figures is presented in Table 6-5.

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*Table 6-6 Performance of bit using SEPT (averages)*

| Rock Type | Bit Type | Annulus (mm) | Bond Strength (kN/mm) | Contact Shear Strength (KPa) | Max Load Achieved (kN) | Overall Stiffness (kN/mm) |
|---|---|---|---|---|---|---|
| Sandstone | 2-Prong | 2.96 | 0.60 | 7330.47 | 150.00 | 128.48 |
| Sandstone | Spade | 2.83 | 0.48 | 5842.97 | 120.00 | 102.35 |
| Shale | 2-Prong | 3.30 | 0.51 | 6036.35 | 126.67 | 101.94 |
| Shale | Spade | 3.10 | 0.34 | 4110.14 | 85.00 | 23.20 |

### 6.6.2 Effect of hole annulus

Borehole annulus is defined as half of the difference between the bolt and hole diameters. As a continuation to the investigation to determine the effect of borehole annulus on support performance, an additional 68 short encapsulated pull tests were conducted under near identical conditions in sandstone and shale roofs. These tests were done using a variety of different sized drill bits in order to attain the necessary annuli. The results from these tests are shown in Figure 6-48.

As can be seen, the results from these tests show that an annulus between 2.5 mm 3.8 mm resulted in the highest bond strengths. Another interesting point is that as the annulus drops below 2 mm, it appears to have a negative effect on the bond strength. This confirms the findings of tests conducted by Hagan (2003) in Australia and the recommendations made by Wagner as far back as 1985.

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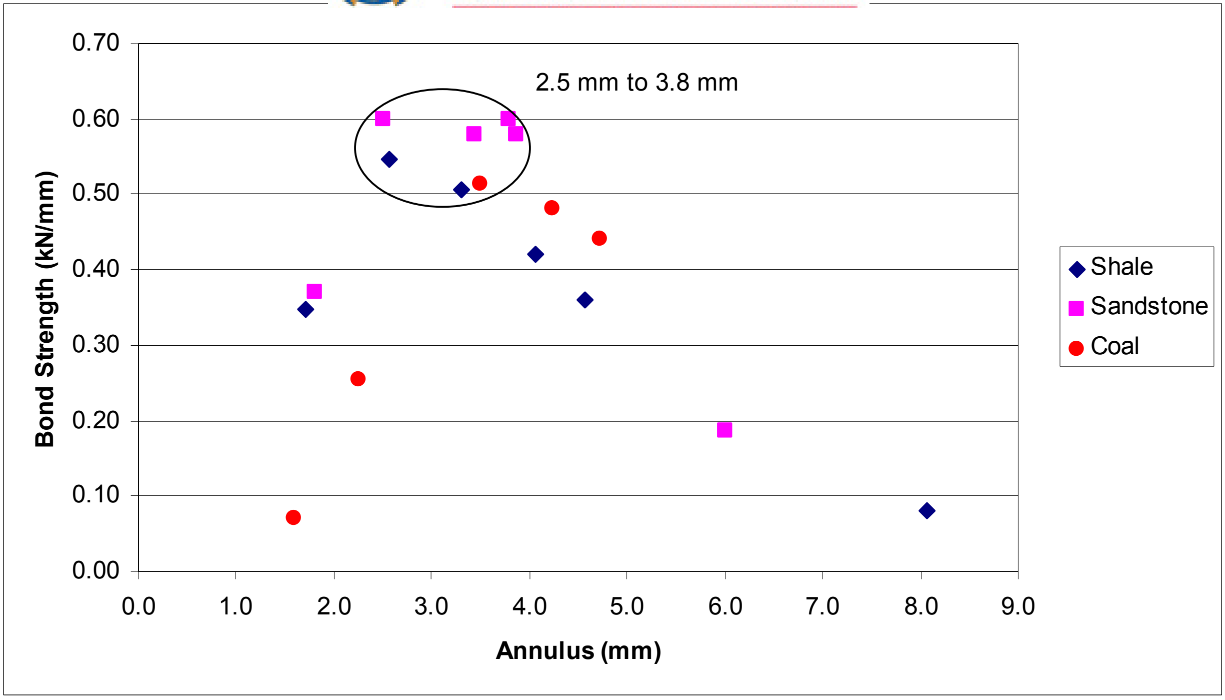


***Figure 6-48 Effect of hole annulus on bond strength***

Note that the annuli in Figure 6-48 are determined from the actual hole and bolt diameter measurements, and not from the bit size. Generally, 24 mm or 25 mm bits with 20 mm roof bolts give an annulus of 2.5 mm and 3.8 mm respectively. It is therefore suggested that these bit sizes should be used with 20 mm roof bolts.

### 6.6.3 Effect of rock types

As has been indicated previously by many researchers, rock type greatly affects support performance. To investigate this effect, a series of pull tests were conducted at different collieries near identical conditions.

Figure 6-49 highlights the very distinct differences between bolt system performances in different rock types. The results clearly show that sandstone produces significantly better results than shale and coal, as was explained in Section 6.5 of this report. From these results it can be concluded that rock type is one of the primary factors influencing support system performance.

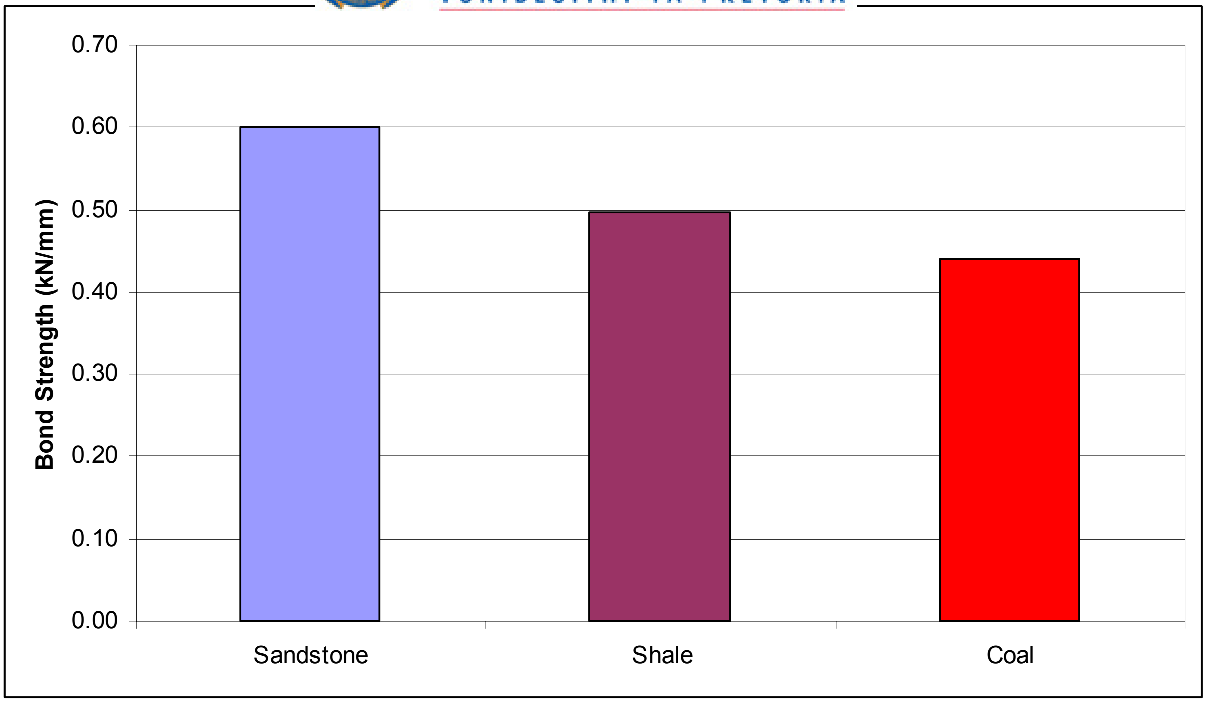


***Figure 6-49 Effect of rock type on support performance***

## 6.7 Quality control procedures for support elements

It is estimated that approximately 6.5 million roof bolts are installed annually in South African collieries (Henson, 2005). Although there are systems available to test the integrity of installed bolts, it is important to ensure that the roof bolts are installed in the best way possible.

There are several factors contributing to the under-performance of roof bolts. These factors should be regularly controlled by systematic quality control procedures.

The factors that can affect the performance of a roof bolt support system can be classified as:

- Direct controllables; and
- Indirect controllables.

The indirect controls are related to suppliers' quality control procedures, such as metallurgical properties of roof bolts, deformation pattern of roof bolts, and chemicals used in the manufacturing process of resin capsules and the consistency of these properties. It is suggested that mining houses should request to examine their suppliers' quality control procedures. It is also suggested that these quality control procedures should comply with ISO standards and that an independent auditor should regularly check for compliance.

The direct controllables can also be divided into three distinct groups (Table 6-7):

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- Support elements;
- Compliance with the design; and
- Quality of installation.

As part of this task of the study, currently available quality control procedures established by the mines in South Africa have been reviewed. These systems are the basis of the quality control procedures presented here.

***Table 6-7 A list of direct controllables***

| **Support elements** | **Installation** | **Compliance with the design** |
|---|---|---|
| Roof bolts | Correct installation cycle | Spacing |
| *Strength of roof bolts* | Correct spinning-holding times | Using correct bolt |
| *Correct length* | Correct insertion of resin | Using correct resin |
| *Correct diameter* | Correct drilling | Correct hole size |
| *Corrosion* | Correct bit size | Correct drill bit |
| *Straightness* | Correct rod and hole length | Correct adjustment of roofbolters |
| Resin | Correct flushing | |
| *Strength* | Correct roof bolt pattern | |
| *Storage* | Correct time-to-installation | |
| *Type* | Correct resin storage | |
| Borehole | | |
| *Diameter and annulus* | | |
| *Straightness* | | |
| *Location and inclination* | | |
| *Length* | | |
| *Roughness* | | |
| Roofbolters | | |
| *Torque* | | |
| *Thrust* | | |
| *Speed* | | |
| Accessories | | |
| *Washer strength* | | |
| *Washer size* | | |
| *Nut strength* | | |
| *Threat type* | | |

From the above Table, the following quality control procedures have been recommended in this thesis.

### 6.7.1 Support elements

| ROOF BOLTS | | | |
|---|---|---|---|
| 1 | Length | General | Roof bolt assemblies are to be supplied in standard lengths (see table below) with the provision available for the supply of non-standard lengths at the request of the client. The tolerance on roof bolt length shall be -5 mm +15 mm. |
| 2 | Profile | Diameter tolerance | The maximum tolerance on roof bolt diameters should be within 0.235 mm. |
| | | Rib height | Should meet the SEPT requirement. |
| | | Rib thickness | Should meet the SEPT requirement. |
| | | Rib distance | Should meet the SEPT requirement. |
| 3 | Straightness | General | Deviation form straight must be within 0.4% of the length of the supplied bolt. |
| 4 | Finish | General | The roof bolt must be free of any grease and defects such as burrs, sharp edged seams, laps or irregular surfaces that may affect its serviceability. |
| 5 | Colour coding | General | Colour coding; the base of the threaded portion or forged head (proximal end) of every roof bolt supplied must be colour coded according to the following table:<br>Nominal roof bolt length (m) - Colour coding:<br>0.6 - Orange<br>0.9 - Yellow<br>1.2 - Blue<br>1.5 - White<br>1.8 - Green<br>2.1 - Pink<br>2.4 – Red |
| 6 | End of bolt | General | The non-threaded end of the roof bolt must be free of burrs and edges that protrude beyond the roof bolt profile. Depending on the requirement of the mine: the non-threaded end of the roof bolt must be formed square by cropping; the threaded end of the roof bolt must be acceptably square to the longitudinal axis of the shank; and must be cropped at the distal end at 45º. |

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| | | | |
|---|---|---|---|
| 7 | Threaded section | General | The threads are to be roll-formed for 120 mm on the bar and when gauged, must be parallel throughout its length. The basic profile of the thread shall conform to the relevant dimensions specified in DIN 405 Part 1: Knuckle Threads. |
| | Run-out | General | In the thread run-out bolt systems, the thread run-out must not exceed three pitches. |
| | Thread Eccentricity | General | Any thread eccentricity of the roof bolt over a thread length of one roof bolt diameter from the thread run-out of the roof bolt measured at any point on the unthreaded shank within a distance of 1.5 roof bolt diameters from the thread run-out must not exceed 0.70 for the 16 mm roof bolt and 0.84 for a 20 mm roof bolt. |
| | Nib bars | General | Any roof bolt with nibs on the threaded section shall, when tested for mechanical performance, not fracture at the cross-section where the nibs are located. |
| | Nut Break Out | General | Any roof bolt supplied with shear pins or other approved breakout facility will have a breakout force for nuts in the range of 90 Nm to 110 Nm for 16 mm and 140 Nm to 170 Nm for 20 mm. |
| 8 | Mechanical Performance (Resin tendons) | Ultimate tensile strength | The ultimate tensile strength of the roof bolt must be at least 15% greater than the yield stress on each tensile test. |
| | | Yield stress | Minimum yield stress shall be 480 MPa. |
| | | Nibs | Any cross-section nibs located on the threaded section of the roof bolt must not fracture before the specified requirements of the bolt when destructively tested. |
| | | Mechanical properties (Laboratory testing) | **16mm resin tendons or equivalent**<br>Maximum strain at 90 kN: 8 millistrain<br>Maximum strain at 100 kN: 12 millistrain<br>Tendon diameter : 16 mm ($\pm$ 0.235 mm)<br>Minimum usable thread length: 100 mm |
| | | | **18mm resin tendons or equivalent**<br>Maximum strain at 140 kN: 13 millistrain<br>Maximum strain at 150 kN: 18 millistrain<br>Tendon diameter 17.3 mm ($\pm$ 0.235 mm)<br>Minimum usable thread length: 100 mm |

| | | | |
|---|---|---|---|
| | | | **20 mm resin tendons or equivalent**<br>Maximum strain at 140 kN: 10 millistrain<br>Maximum strain at 150 kN: 13millistrain<br>Tendon diameter 20 mm (± 0.235 mm)<br>Minimum usable thread length: 100 mm |
| | | Mechanical properties (Underground SEP testing) | The maximum load achieved must not be less than:<br>125 kN for 20 mm roof bolts<br>100 kN for 18 mm roof bolts<br>85 kN for 16 mm roof bolts<br><br>The minimum system stiffnesses must be:<br>20 mm bolt 60 kN/mm<br>18 mm bolt 50 kN/mm<br>16 mm bolt 40 kN/mm |
| 9 | Mechanical Performance (Mechanical bolts) | Underground testing | Performance during underground testing |
| | | | Minimum pull-out load |
| | | | Units must achieve 70 kN of pull-out load. |
| | | | Maximum deformation must not exceed 1.2 times the average deformation attained by the control installations. |
| | | Maximum deformation | Mechanically anchored roof bolts should be provided by Rock Engineering in control installations. |
| | | Control installation | Roof bolts and studs shall comply with the following specifications: |
| | | Specifications | They must have Bail-type or Regular shells, and be equipped with crimp nuts failing at torque equivalent to a pre-tension of 20 kN to 40 kN or Bail-type shells with forged head. |
| | | | Maximum strain at 70 kN: 4 millistrain<br>Maximum strain at 80 kN: 5 millistrain<br>Minimum tendon diameter: 14.5 mm<br>Minimum usable thread length: 100 mm |
| 10 | Washers | General | Washers must be manufactured from steel and must be a minimum of 120 mm x 120 mm square. |
| | | Surfaces | All surfaces must be free of burrs and sharp edges |
| | | Holes | Holes in the dog-eared portion of washers must not be closer then 3 mm to the edge of the washer. |
| | | Shape | Washer plates must be square or round type (deformed or ribbed and with or without dog-ears). |

| | | | |
|---|---|---|---|
| | | Specifications | **For use with 18 mm tendons**:<br>Washers for use with 18mm tendons must meet the following specifications:<br>• Maximum displacement at 140 kN: 13 mm<br>• Maximum displacement at 150 kN: 18 mm |
| | | | **For use with 20 mm tendons:**<br>Washers for use with 20 mm tendons must meet the following specifications:<br>• Maximum displacement at 140 kN: 10 mm<br>• Maximum displacement at 150 kN: 13 mm |
| | | | **For use with all other tendons** |
| | | | Washers for use with all other tendons must meet the following specifications:<br>Maximum displacement at 90 kN: 8 mm<br>Maximum displacement at 100 kN: 12 mm |
| 11 | Nuts | General | Nuts must be of hexagon steel. The dimensions across the flats shall be 24 mm for a 16 mm roof bolt and 32 mm for a 20 mm roof bolt. |
| | | Processing | All nuts are to be cold forged from steel and should be heat treated to provide the required mechanical properties. |
| | | Compliance | Nuts must comply with the relevant requirements for eccentricity and tilt as in SABS 135. |
| | | Compliance | The threads must conform to DIN 405: Part 1 as applicable to nut size. |
| | | Manufacturing process | All nuts must be manufactured from a higher grade steel than the tendon and washer, the steel grade to be a minimum of grade 6. When tested, all nuts must achieve a surface hardness of Vickers 220 to 302HV. |
| | | Performance | When tested to destruction in the laboratory the nut must not fail in any way before the ultimate strength of the tendon is exceeded. The Rock Engineering Department may from time to time call for destructive testing as it sees fit. For routine quality control tests, nuts used with the following tendons must not fail at the following minimum loads: |
| | | | a) Smooth bar (mechanical anchors): 85 kN |
| | | | b) 16mm tendons 110 kN |
| | | | c) 18mm and 20 mm tendons 170 kN |

| | | | |
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| | | Load indicators | One in each ten bolts shall be supplied with a device capable of visually indicating that an installation has been adequately pre-tensioned. During static laboratory testing (not spun or torqued) the indicators must fail at a load of between 45 kN and 55 kN (4.5 to 5.5 tonnes). |
| | | Nut break out | The nut break out facility must operate at the torque range values detailed below:<br>• Bolt Length 0. 9m, 1.2 m - 70 Nm to 90 Nm<br>• Bolt Length 1.5 m, 1.8 m, 2.1 m - 110 Nm to 140 Nm |
| 12 | Drill bits | General | Only the following (nominal) size drill bits should be supplied to mine for the purpose of drilling holes to install ground support material:<br><br>For resin tendon applications:<br>• For 16 mm and 18 mm roof bolts: 22 mm<br>• For 20 mm roof bolts: 23.5 mm<br>• For cable anchor applications: 36mm<br>• For mechanically anchored roof bolts: 36 or 38mm<br><br>All drill bits (borers) must be manufactured with a tolerance of -0/+0.25 mm. |

| ROOFBOLTERS | | | |
|---|---|---|---|
| 1 | General | | Roofbolters should be regularly maintained, and have the following specifications (note that these specifications are to achieve rough hole profiles, and if necessary, they can adjusted to requirements): |
| 2 | Specifications | Torque | The torque on the roofbolter must be between 220 kN to 250 kN. |
| | | Thrust | The thrust on the roofbolter must be between 12 kN to 18 kN. |
| | | Speed | The speed of the roofbolter must be 350 rpm to 550 rpm. |

| RESIN | | | |
|---|---|---|---|
| 1 | General | Capsule | All resin must be supplied in capsule form. |
| | | Compliance | All resin capsules used must conform to SABS 1534:2002. |

| | | | |
|---|---|---|---|
| | | Information required | The following information must be shown clearly on each box of resin:<br>a) Capsule dimensions<br>b) Expiry date<br>c) Batch number<br>d) Spin and hold times |
| 2 | Capsule Size | Tolerance | Capsules must be 19 mm ± 0.5 mm in diameter for use with 16 mm bolts and 23 mm ± 0.5 mm in diameter for use with 20 mm bolts. The tolerance on supplied length must be nominal ordered length +10 /-5 mm when measured between the crimped ends. |
| 3 | Colour Coding | Colour coding | Resin types must be identified by a self-colour coding as given below:<br>• Fast Set – Red<br>• Slow Set – Yellow |
| 4 | Shelf Life | General | All resins must retain their ability to conform to the performance requirements of this specification and retain sufficient rigidity for insertion with a capsule-loading tube for a minimum period of six months when they are stored in accordance with the manufacturer’s instructions. |
| 5 | Packaging | General | All packing must be capable of withstanding transportation, handling and storage, and general handling associated with the mining environment. |
| | | Information required | Each package must be identified with the manufacturer’s name, type of resin, size of capsule, and quantity of capsules, and be of a colour consistent with the resin-type colour code specified above. |

| | | | |
|---|---|---|---|
| | | Information display | The following additional information must be displayed on all packages in a position that is visible when the packages are stacked:<br>a. Capsule dimensions<br>b. Expiry date<br>c. Batch number<br>d. Nominal mixing and holding time<br>e. Shelf life and storage instructions<br>f. Date of manufacture<br>g. Batch and time reference<br>h. Manufacturer's identification<br>i. The symbols, risk and safety phrases as required under the Safety Regulations<br>j. Remedial measures in the event misuse/accident<br>k. Installation procedure taking into account applicable regulations. |
| 6 | Gel and Setting Time | General | Gel setting times for different spinning speeds and temperatures should be clearly indicated on the box. |
| 7 | Bond Strength and System Stiffness | Performance | When tested in SEPT, the minimum bond strength between roof bolt and resin must be 95 kN for 16 mm bar, 120 kN for 18 mm bar and 140 kN for 20 mm bar. The minimum system stiffness must be 60 kN/mm measured between loads of 40 kN and 80 kN, based on underground pull tests. |
| 8 | Uniaxial Compressive Strength (UCS) | Performance | The UCS of the resin must be greater than 60Mpa when it is measured at least 24 hours after preparation of the test specimens. The number of tests should be determined from the methodology described in this report. |
| 9 | Elastic Modulus | Performance | The elastic modulus of the resin must not be less than 10GPa when it is measured 24 hours after preparation of the test specimens. The required number of tests should be determined from the methodology described in this report. |
| 10 | Creep | Performance | The creep of the resin must be no more than 0.12% when it is measured 24 hours after preparation of the test specimens. |
| 11 | Shear strength | Performance | Must meet the SEPT requirements.<br>The maximum load achieved must not be less than:<br>125 kN for 20 mm roof bolts<br>100 kN for 18 mm roof bolts<br>85 kN for 16 mm roof bolts |



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| | | | |
|---|---|---|---|
| | | | The minimum system stiffnesses must be:<br>20 mm for bolt 60 kN/mm<br>18 mm for bolt 50 kN/mm<br>16 mm for bolt 40 kN/mm |

| **ROUTINE TESTS** | | | |
|---|---|---|---|
| 1 | Roof bolts | Mechanical properties | At least 5 bolts from each batch supplied to the mine should be tested in the laboratory. |
| | | Length | As a routine test, one roof bolt in every 200 produced must be checked for length using a measuring tape. |
| | | Diameter | As a routine test, one roof bolt in every 200 produced must be checked for diameter using a Vernier. |
| | | Straightness | As a routine test, one roof bolt in every 200 produced must be checked for straightness using an appropriate gauge. |
| | | Rib height | As a routine test, one roof bolt in every 200 produced must be checked for rib height using a Vernier. |
| | | Washer | At least 5 from each batch should be tested in the laboratory. |
| | | Thread | As a routine test, one roof bolt in every 200 produced must be checked for thread. |
| | | Nuts | At least 5 from each batch should be tested in the laboratory. |
| 2 | Resin | Length | As a routine test, one resin in every 10 boxes produced must be checked for length using a Vernier. |
| | | Diameter | As a routine test, one resin in every 10 boxes produced must be checked for diameter using a measuring tape. |
| | | Mechanical properties | At least 5 from each batch should be tested underground using short encapsulated pull tests. |
| 4 | Roofbolters | Torque, thrust and speed | As a routine test, roofbolter's torque, thrust and speed must be checked once every month. |

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### 6.7.2 Compliance with the design

Compliance with the design should be checked underground at least once every fourth week. The following parameters should be measured and recorded:

- Spacing of roof bolts using a simple measuring tape;
- The use of correct bolt type;
- The use of correct resin type;
- Correct hole size using a borehole micrometer;
- The use of the correct drill bit; and
- Correct adjustment of torque, thrust and speed of roofbolters using a torque wrench, load cell and tachometer, respectively.

### 6.7.3 Installation

Underground support installation is one of the most important aspects of support performance. The following parameters should be measured and recorded every fourth week using the appropriate instruments, where necessary:

- Correct installation cycle;
- Correct spinning-holding times;
- Correct insertion of resin;
- Correct drilling;
- Correct bit size;
- Correct hole size;
- Correct rod length and hole length;
- Correct flushing;
- Correct roof bolt pattern;
- Correct time-to-installation; and
- Correct resin storage.

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## 6.8 Conclusions

Although a considerable amount of time was spent on the effect of the roofbolters on the performance of support systems, few trends could be observed in the parameters influencing the support performance. The study showed that there are no standards in South Africa for the parameters investigated (speeds, torque, and thrust). Underground testing showed that the variations in the parameters are greater than was previously believed. No correlation between the hole profiles and the parameters investigated could be discerned.

Nevertheless, this indicates that in South Africa, the installation quality of bolts varies significantly. Irrespective of design, the bolts are installed in completely different manners. Unfortunately, there is no data available on the relationship between roof collapses and the quality of bolt installation. It is therefore impossible to determine empirically which support installation performs the best. This highlights a need for the best equipment performance for the best support installation to be investigated in detail. Such a study would assist in reducing the falls of ground and, therefore, the rock-related casualties in South African collieries. However, experience gained during the underground experiments showed that such work can only be done in a more controlled environment, such as with the laboratory.

Five important elements of a bolting system have been identified. The impacts of those elements were qualified through short encapsulated pull tests.

The performance of roof bolts that are currently supplied to South African mines was also investigated by a series of short encapsulated pull tests. The results indicated that bolts from all four manufacturers showed almost identical results in sandstone, while in shale the results were dissimilar.

To determine whether variations in the profile of bolts supplied by the different manufacturers could account for the differences in performance, the bolt-core diameters and rib diameters from different bolt manufacturers in South Africa were measured.

The parameters that determine the contact strength between bolt and resin are rib-height, spacing between the ribs, and the rib angle. An investigation was conducted into the dimensions of roof bolts that are used currently. The results showed insignificant differences between the parameters that determine the bolt profile of South African roof bolts. Owing to the physical similarity between the bolts studied, it was not possible to determine the influence of these parameters.

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The effect of rib angle was investigated and the results of a literature search showed that, as the rib angle increases away from normal to the bolt axis, so the pull-out load of the bolt decreases. It is therefore suggested that, in order to achieve relatively high pull-out loads, low rib angles on the bolts are required. This was confirmed by laboratory tests on different bolts with different rib angles in Australia (O'Brien, 2003). However, it is noted that lowering the rib angle may result in poor resin mixing performances.

Using a conceptual model to determine the effect of bolt profiles, it is shown that maximum pull-out loads can be achieved between the resin and roof bolt when:

- The ribs are relatively high;
- The distance between the ribs is relatively low; and
- The ribs are relatively thick.

The performance of resins that are currently being used in South African collieries was also investigated by means of short encapsulated pull tests. The results indicated that in sandstone the resin types from the two different manufacturers performed similarly. However, the strength of slow (5/10-minute) resins from both manufacturers was much lower than that of fast resins. It is concluded that in the majority of pull tests, failure took place at the rock-resin interface, indicating that the rock failed before the resin shear strength had been reached. It is therefore suggested that the strength of resin currently being used in South Africa is adequate. However, the stiffness of the system of which resin is a part should be determined by short encapsulated pull tests.

Again, the conceptual model developed to determine the effect of resin in the support system concluded that the failure characteristics of a roof bolting system will be determined by the shear strength of bolt, resin, and rock.

- The failure will take place at the resin-rock interface when the shear strength of the rock is lower than the resin (rock will fail).
- The failure will take place at either the resin-rock or resin-bolt interface when the resin shear strength is the lowest in the system.
- When the resin shear strength is the lowest in the system, the failure will be determined by the roughness of the hole and the bolt profile.

The test results showed that the reinforcing system using bolts from all four manufacturers performed almost identically in sandstone, but performed in different ways in the other rock

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types. The bolts from Manufacturer "A" performed slightly better in coal and shale rock types than the bolts from other manufacturers.

In order to investigate the effect of bit types, a series of short encapsulated pull tests were conducted. The results showed that the 2-prong bit outperformed the spade bit in sandstone and shale rock types. However, the average hole annuli obtained from the 2-prong bit were always greater than the spade bit. It is thought that this is because 2-prong bits drilled a rougher hole profile. Both the stiffness and the maximum load obtained from the 2-prong bits were greater than for the spade bits. These findings suggest that 2-prong bits are more effective in collieries than spade bits are.

The effect of hole annulus was also investigated. The results show that an annulus between 2.5 mm 3.8 mm resulted in the most effective bond strengths. Another interesting point is that as the annulus drops below 2 mm, it appears to have a negative effect on the bond strengths.

The effect of wet and dry drilling was also investigated by means of short encapsulated pull tests. The results showed that bond strengths and overall support stiffnesses are greater with the use of the wet drilling in all three resin types. The reason for this was not determined but is probably related to the surface condition of the holes and its influence on the adherence of the resin to the rock.

Tensioned versus non-tensioned bolts is one of the most discussed topics in roof bolting. A number of papers have been published on this topic in Australia and the US. An additional 25 short encapsulated pull tests were conducted to determine the effect of tensioning on bond strength. The results showed that non-tensioned roof bolts achieved significantly higher bond strengths than the tensioned bolts in sandstone and shale roofs. Similarly, the overall support stiffness of non-tensioned roof bolts was significantly greater than that of the tensioned roof bolts. This finding may be significant and therefore the effect of tensioning and non-tensioning on overall support system performance should be investigated in a control environment.

The effect of rock type on support performance was also investigated by means of a series of short encapsulated pull tests. The results from these tests highlight the very distinct differences between bolt system performances in different rock types. Sandstone was shown in the tests to produce significantly better results than shale and coal. From these results it can be concluded that rock type is one of the primary factors influencing the support system performance.

An investigation into the quality control procedures of support systems was also conducted. Quality control procedures for compliance with the design, support elements and quality of

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installation are presented in the thesis. Recommendations for improving quality control measures and for developing testing procedures for bolt system components, installation quality and resin performance are provided.

Most importantly, similar to stress regime, geology and roof characteristics presented in the previous Chapters, there is a significant variation in the performance of support systems using different support components in different geotechnical environments. Therefore, it is concluded that a deterministic approach is not adequate for a roof bolting system design in such a complex system. A probabilistic approach is required in order to take all these variations into account.

# Chapter 7.0

# Roof support design methodology

## 7.1 Introduction

In order to develop an engineering design, it is essential to understand the roof and support behaviour and the interaction between them. A detailed analysis of the data given in the previous Chapters was therefore conducted and the results are presented in this Chapter.

As demonstrated in the previous Chapters that in an underground environment rock and support properties and performances can vary significantly within a short distance. The roof stability is strongly dependent on these varying properties of roof-support system. These properties can be described using deterministic and/or probabilistic models. Deterministic models typically use a single discrete descriptor for the parameter of interest. Probabilistic models however describe parameters by using discrete statistical descriptors or probability distribution (density) functions. Therefore, a roof support design methodology based on probabilistic approach has been developed and presented in this Chapter. It is considered that for real world roof support problems, the values of input parameters are not constant and a single safety factor cannot be used.

It is however not intended to present a complete and rigorous treatment of the fundamentals of probabilistic design approach, therefore the formal theory of probability is summarised and a functional description is presented.

## 7.2 Support design based on a probabilistic approach

In traditional deterministic (calculation of a single safety factor) roof bolt design methodologies, the input parameters are represented using single values. These *certain* values are described typically either as “best guess” or “worse case” values. However, investigations into the roof and roof bolt behaviour presented throughout this thesis suggest that the input parameters, including the mining geometries, rock and support properties can vary significantly within a few meters in a panel and also from one support product to another. This is the fundamental principal of probabilistic design approach, which is the recognition of that these factors which govern the roof stability and support performance exhibit some degree of natural uncertainty. Ideally, this uncertainty should be accounted for in the design method. While deterministic approaches provide some insight into the underlying mechanisms, they are not well-suited to making

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predictions to roof support decision-making, as they cannot quantitatively address the risks and uncertainties that are inherently present. In a probabilistic design method however, the stochastic nature of the input parameters are included and therefore, it is possible to quantitatively represent uncertainties thus the resulting probability of failures. Dealing with probabilities of failure rather than safety factors means that it is acknowledged that realistically there is always a finite chance of failure, although it can be very small.

### 7.2.1 Rules of probability

The first rule of probabilistic approach is that, by convention, all probabilities are numbers between 0 and 1. A probability of 0 indicates an impossible event, and a probability of 1 indicates an event certain to happen. Most events of interest have probabilities that fall between these extremes.

The second rule states that, if two events are dependent (i.e., knowing the outcome of one provides information concerning that the other will occur), then the probability that both events will occur is given by the product of their combined probabilities. Assume, $E_1$ and $E_2$ are two events and the event that both $E_1$ and $E_2$ occur is described as $P[E_1E_2]$ and is calculated:

$$P[E_1E_2] = P[E_1]xP[E_2/E_1] \quad [7\text{-}1]$$

where $P[E_2/E_1]$ is the probability of $E_2$ occurring given that $E_1$ has taken place. If $E_1$ and $E_2$ are independent, that is the occurrence of one does not affect the probability of occurrence of the other, indicating that the probability of two independent events occurring is the product of their individual probabilities:

$$P[E_2/E_1] = P[E_2] \quad [7\text{-}2]$$

$$P[E_1E_2] = P[E_1]xP[E_2] \quad [7\text{-}3]$$

Probabilistic methods have long been used mainly in civil and other engineering disciplines. Examples of this can be found where probabilistic design methods are used almost routinely to assess the failure probability of building structures and rock slopes.

### 7.2.2 Methodology of probabilistic approach

The general methodology of probabilistic approach assumes that the load ($L$) and the strength ($S$) of a structure can be described by two probability density functions, respectively, as shown in Figure 7-1. The respective mean and standard deviations of each distribution is denoted $m_S$ and $s_s$ for the strength, and $m_L$ and $s_L$ for the load. From Figure 7-1 it can be seen that the two curves overlap meaning that there exist values of strength which are lower than the load, thus implying that failure is possible in this overlap area. In a purely deterministic approach using only the mean strength and load, the resulting factor of safety would have been significantly larger than unity which implies stable conditions.

To be able to calculate the probability that the load exceeds the strength of a construction element, it is common in Civil Engineering to define a safety margin, $SM$, as

$$SM = S - L \tag{7-4}$$

The safety margin is one type of performance function which is used to determine the probability of failure. The performance function is often denoted $G(X)$, hence:

$$G(X) = S(X) - L(X) \tag{7-5}$$

where $X$ is the collection of random input parameters which make up the strength and the load distribution, respectively. An alternative formulation of the performance function which is often used in geomechanics involves the factor of safety, $F_S$. Failure occurs when $F_S$ is less than unity, hence the performance function is defined as:

$$G(X) = F_S - 1 \tag{7-6}$$

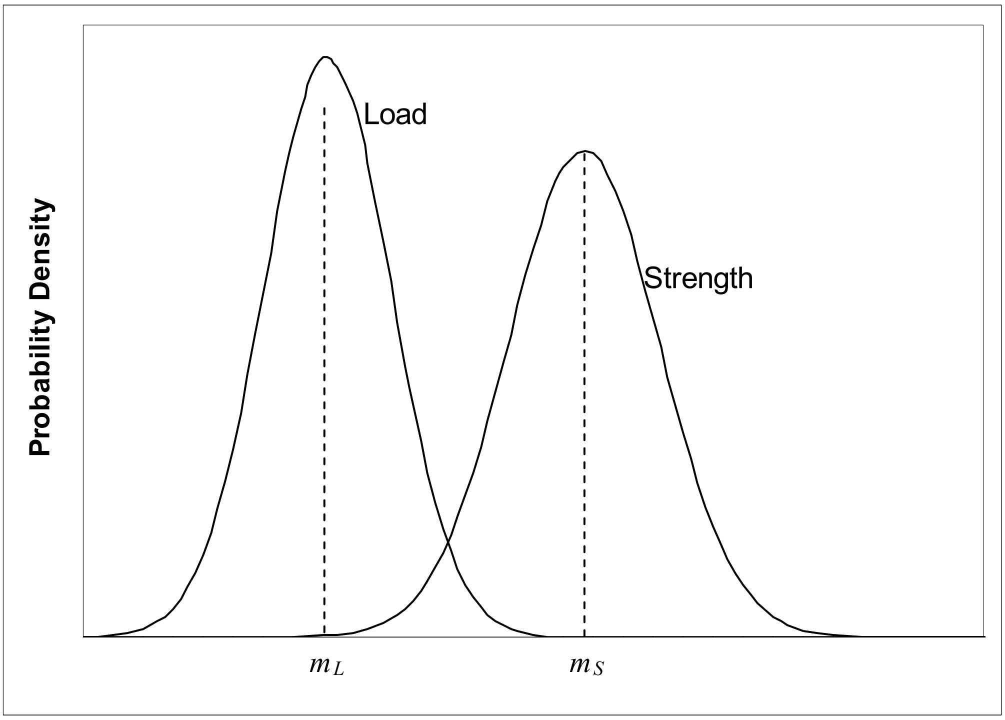


***Figure 7-1 Hypothetical distribution of the strength and the load***

The probability density function (PDF) for the safety margin is illustrated in Figure 7-2. This Figure indicates that failure occurs when the safety margin is less than zero. The probability of failure (PoF) is the area under the density function curve for values less than zero, as shown in

Figure 7-2. The reliability of a structure, on the other hand, is defined as the probability that the construction will *not* fail. The same concept applies to any performance function.

Assuming that the performance function can be expressed according to either Equation [7-5] or [7-6] and that the strength and load distributions can be defined, using a 3-level analysis (Level 1, Level 2 and Level 3), probability of failure can be calculated. A Level 1 analysis is basically a deterministic analysis, i.e. only one parameter value is used for every variable. In a Level 2 analysis, each stochastic variable is characterized by two parameters, the mean and the standard deviation, as described above. A Level 3 analysis is the most complete and sophisticated method of assessing the probability since the exact statistical characteristics of all variables are taken into account and the joint probability density functions are calculated. Level 3 analysis is fairly uncommon since it often is very difficult to describe and quantify the "joint probability density functions" (Mostyn and Li, 1993).

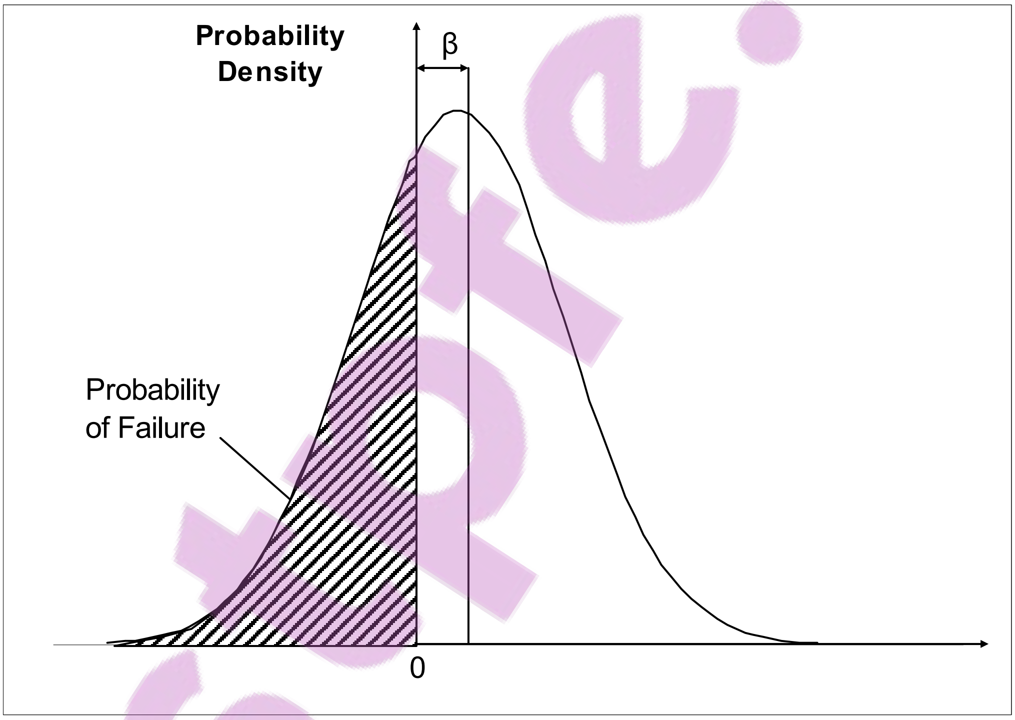


***Figure 7-2 Hypothetical distribution of the safety margin, SM.***

In practical Civil Engineering design, Level 2 analysis is most commonly used and found acceptable (Sjoberg, 1996). Level 2 analysis is also adopted in this thesis. In this approach, the probability of failure is evaluated using a reliability index, $\beta$, defined in terms of the mean and the standard deviation of the trial factor of safety:

$$\beta = \frac{m_G - 1}{s_G} \qquad [7\text{-}7]$$

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where $m_G$ and $s_G$ are the mean and standard deviation of the performance function, respectively. The reliability index (RI) is thus a measure of the number of standard deviations separating the mean factor of safety from its defined failure value of 1.0, Figure 7-2. It can also be considered as a way of normalising the factor of safety with respect to its uncertainty. When the shape of the probability distribution is known, the reliability index can be related directly to the probability of failure. In Civil Engineering, especially in building construction design, the reliability index has been linked to safety classes for buildings. This will be discussed in detail further in this Chapter.

Exact solutions for calculating the failure probability is only possible for simple cases. The performance function contains several variables describing the load and strength and is therefore often non-linear, which prohibits exact analytical solutions. A commonly used approximate method is the first-order-second-moment method (FOSM) in which the performance function is approximated by a polynomial expansion into a linear expression. Using a linear expression, the mean and standard deviation of the performance function and also the reliability index can be calculated using standard statistical formulae (Mostyn and Li, 1993). The resulting distribution of the performance function can be assumed to be a normal distribution, according to the central limit theorem (Kreyszig, 1988). Consequently, the resulting failure probability can be calculated as $\Phi(-\beta)$ where $\Phi$ is the standardised normal distribution which can be found tabulated (Kreyszig, 1988). The FOSM provides no information about the shape of the probability density function. To estimate any probability, the shape of the probability distribution of the output has to be assumed. This assumption of, typically, a normal or a lognormal distribution introduces a source of inaccuracy.

An alternative technique is the point estimate method (PEM), which is based on the precept that a probability distribution can be represented by point estimates. In this method the performance function is evaluated $2^N$ times (N being the number of input variables) to obtain the mean and standard deviation of the performance function (Rosenblueth, 1975). This method is very simple for two-three variables and does not require extensive mathematical derivations, however, become impractical for large numbers of input parameters.

Another slightly different definition of the reliability index is that given by Hasofer and Lind (1974), in which the reliability index is defined as the shortest distance from the origin and to the boundary of the limit state. The function is the limit state is determined from the performance function by transforming to statistically uncorrelated variables. The reliability index $\beta$ can then be determined iteratively. Hasofer-Lind's (1974) method is common in building construction design but has limitations regarding how complex the performance function can be to do the transformation to uncorrelated parameter space.

All of the above methods are analytical means of determining the reliability index from a number of stochastic variables which make up the performance function. In cases where the performance function is complex and contains a large number of variables, a simulation technique can instead be used. The most common simulation technique is the Monte Carlo method. In this method, the distribution functions of each stochastic variable must be known. From each distribution, a parameter value is sampled randomly and the value of the performance function calculated for each set of random samples. If this is repeated a large number of times, a distribution of the performance function is obtained. The probability of failure can be calculated as the ratio between the number of cases which failed and the total number of simulations. Alternatively, the mean and standard deviation of the performance function distribution (factor of safety) can be calculated to yield the reliability index from which the failure probability can be determined using tabulated values for the standardised normal distribution (Kim et al. (1978); Mostyn and Li (1993)).

Monte Carlo simulation is thus a procedure in which a deterministic problem is solved a large number of times to build a statistical distribution. It is simple and can be applied to almost any problem and there is practically no restriction to the type of distribution for the input variables. The drawback is that it can require substantial computer time. This becomes especially important when relatively small probabilities are expected and hence many iterations are required to obtain a reliable measure of the tails of the distribution. To overcome this, more efficient Latin Hypercube sampling technique has been developed. In this method, stratified sampling is used to ensure that samples are obtained from the entire distribution of each input variable. This results in much fewer samples to produce the distribution of the performance function, in particular for the tails of the distribution (Nathanail and Rosenbaum, 1991; Pine, 1992). With today's powerful computers and widely available softwares, such as RiskAMP (utilised in this thesis) and @RISK, computational time has become less of a problem and Monte Carlo methods prevail as the most common simulation techniques.

In general, the implementation of Monte Carlo method involves:

- Selection of a model that will produce a deterministic solution to a problem of interest.
- Decisions regarding which input parameters are to be modelled probabilistically and the representation of their variabilities in terms of probability distributions.
- Repeated estimation of input parameters that fit the appropriate probability distributions and are consistent with the known or estimated correlation between input parameters.
- Repeated determination of output using the deterministic model.
- Determination of the probability density function of the computed output.

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As mentioned above, the fundamental to the Monte Carlo method is the process of explicitly representing the uncertainties by specifying inputs as probability distributions. By describing the process as a probability distribution, which has its origins in experimental/measurement *continuous* data, an outcome can be sampled from the probability distributions, simulating the actual physical process/measurement.

This process requires a collection of actual measurements and determining the best fits to the data using the goodness of fit tests (GOF). GOF tests measure the compatibility of a random sample with a theoretical probability distribution function. Three most common GOF tests are:

- Kolmogorov-Smirnov
- Anderson-Darling
- Chi-Squared

The details of the probability distributions, GOF tests and random selection of design parameters are given in Section 7.6.

#### 7.2.3 Required number of runs in Monte Carlo simulation

Probabilistic analysis using the Monte Carlo simulation involves many trial runs. The more trial runs used in an analysis, the more accurate the statistics will be. The number of required Monte Carlo trials is dependent on the desired level of confidence in the solution as well as the number of variables being considered (Harr, 1987), and can be estimated from:

$$N_{mc} = \left[ \frac{d^2}{4(1-\varepsilon)^2} \right]^m \qquad [7\text{-}8]$$

where $N_{mc}$ = number of Monte Carlo trials, $d$ = the standard normal deviate corresponding to the level of confidence, $\varepsilon$ = the desired level of confidence (0 to 100%) expressed in decimal form; and $m$ = number of variables.

The number of Monte Carlo trials increases geometrically with the level of confidence and the number of variables. For example, if the desired level of confidence is 90%, the normal standard deviate will be 2.71, the number of Monte Carlo trials will be 68 for one variable, 4,575 for two variables, and 309,445 for three variables. Theoretically, for a 100% level of confidence, an infinite number of trials would be required.

For practical purposes, the number of Monte Carlo trials is usually in the order of thousands. This may not correspond to a high level of confidence when multiple variables are being considered; however, the statistics computed from the Monte Carlo simulations are typically not very sensitive to the number of trials after a few thousands trials (Allen et al., 2002).

### 7.2.4 Acceptable probability of stability

Another important consideration in using the probabilistic approach is to use an *acceptable* PoF in the design.

Vrijling and van Gelder (1998) defined the following three kinds of limit states to construct a breakwater and recommended probability of failures depending on the failure characteristics:

i) Ultimate Limit States (ULS), describing immediate collapse of the structure.
ii) Serviceability Limit States (SLS), describing loss of function of the structure without collapse
iii) Accidental Limit States (ALS), describing failure under accident conditions (collision, explosions).

Vrijling and van Gelder (1998) stated that usually low PoF required for ULS compared to SLS and ALS in which the effects of failure are easily reversed.

Vrijling and van Gelder (1998) developed the following classification and Table 7-1 to be used in the design of vertical breakwaters considering the probability of loss of life due to failure of the structure:

- Very low safety class, where failures implies no risk to human injury and very small environmental and economic consequences.
- Low safety class, where failures implies no risk to human injury and some environmental and economic consequences.
- Normal safety class, where failures implies risk to human injury and significant environmental pollution and high economic or political consequences.
- High safety class, where failures implies risk to human injury and extensive environmental pollution and high economic or political consequences.

***Table 7-1 Acceptance probability of failures for different safety class (after Vrijling and van Gelder, 1998)***

| Limit State Type | Design Probability of Failure | | | |
|---|---|---|---|---|
| | Very Low | Low | Normal | High |
| SLS / ALS | 40% | 20% | 10% | 5% |
| ULS | 20% | 10% | 5% | 1% |

Form the above Table it is evident that even in ultimate limit state 10 to 5 per cent probability is acceptable for low to normal safety classes.

The probabilities used in the design of open cast slopes are discussed with Priest and Brown (1983) and Pine (1992), who defined acceptance criteria according to Table 7-2.

This Table indicates that for benches, probability of failure of around 10 per cent is accepted, whereas for an overall slope, a failure probability of less than 1 per cent would be more suitable.

***Table 7-2 Acceptance criteria for rock slopes (after Priest and Brown, 1983; Pine, 1992)***

| Category and Consequences of failure | Example | Reliability Index ($\beta$) | Probability of Failure |
|---|---|---|---|
| 1. Not serious | Non-critical benches | 1.4 | 10% |
| 2. Moderately serious | Semi-permanent slopes | 2.3 | 1 – 2 % |
| 3. Very Serious | High/permanent slopes | 3.2 | 0.3% |

A design criteria based on probability of failure is also recommended for Western Australian open cast mines, Table 7-3. These design criteria have been developed from a combination of DME assessment of open cast mines in Western Australia and a selection of published literature.

Similarly, this Table suggests a probability of failure of 1 per cent as acceptable in serious slopes. This decreases to 0.3 per cent in populated areas where the slopes are near public infrastructures.

Based on these previous experiences, the probabilistic design criteria presented in Table 7-4 is tentatively suggested for roof bolting system design. It is however recommended that this design criteria should be evaluated before fully implemented in underground coal operations.

***Table 7-3 Examples of design criteria for open pit walls (after DME, 1999)***

| Wall class | Consequence of Failure | Design Probability of Failure | Pit wall examples |
|---|---|---|---|
| 1 | Not Serious | Not applicable | Walls (not carrying major infrastructure) where all potential failures can be contained within containment structures. |
| 2 | Moderately Serious | 10% | Walls not carrying major infrastructure. |
| 3 | Serious | 1% | Walls carrying major mine infrastructure (e.g. treatment plant, ROM pad, tailings structures). |
| 4 | Very Serious++ | 0.30% | Permanent pit walls near public infrastructure and adjoining leases. |

+ Potential failures have been defined as those modes of pit wall failure that have a POF of greater than 10%.
++ Where a mutually acceptable agreement to allow mining cannot be made between the mining company and the "owner" of the adjoining structure or plot of land. Note that a higher standard of geotechnical data is required for the design of category 3 and 4 slopes compared to category 1 and 2 slopes.

***Table 7-4 Suggested design criteria for the roof bolting systems***

| Roof class | Risk Category | Reliability index ($\beta$) | Design Probability of Failure | Example |
|---|---|---|---|---|
| 1 | Moderately Serious | 1.4 | 5% | Short term requirement (< 1 year), personnel access partially restricted |
| 2 | Serious | 2.3 | 1% | Medium term requirement (1 - 5 years) personnel access partially restricted |
| 3 | Very Serious | 3.2 | 0.3% | Long term requirement (> 5 years) no personnel access restrictions |

In Civil Engineering, probabilistic design has advanced to the stage that virtually all building regulations are based on a probabilistic approach. The development has not yet reached this point in the field of geomechanics. One of the reasons for this is the difficulty associated with describing a rock mass quantitatively and defining a model which describes both the strength and the load acting on rock. This requires knowledge of roof failure mechanisms and a model which describes how failure occurs. The following sections of this Chapter aim at developing a deterministic model of failure mechanisms and a load/strength relationship to be used to develop a probabilistic design methodology for coal mine roof support design.

## 7.3 Roof behaviour and failure mechanism

In order to develop a realistic roof behaviour model, data presented in Chapters 3 and 4 was analysed in detail. A total of 55 intersection and roadway measurements from depths of 32 m to 170 m situated in significantly different geotechnical environments were analysed in terms of height and magnitude of instabilities in the roof. The aim of this analysis was to:

1. establish at what heights the instabilities took place,
2. how these instabilities can be supported, and
3. establish a roof behaviour based on the magnitudes of deformations.

The results obtained from the height of instabilities are presented in Figure 7-3. This figure shows that the maximum measured height of instabilities in South African collieries is limited to 2.5 m into the roof, and there is no evidence of a substantial increase in the height of instabilities, as is the case in some overseas coal mines, Figure 7-4.

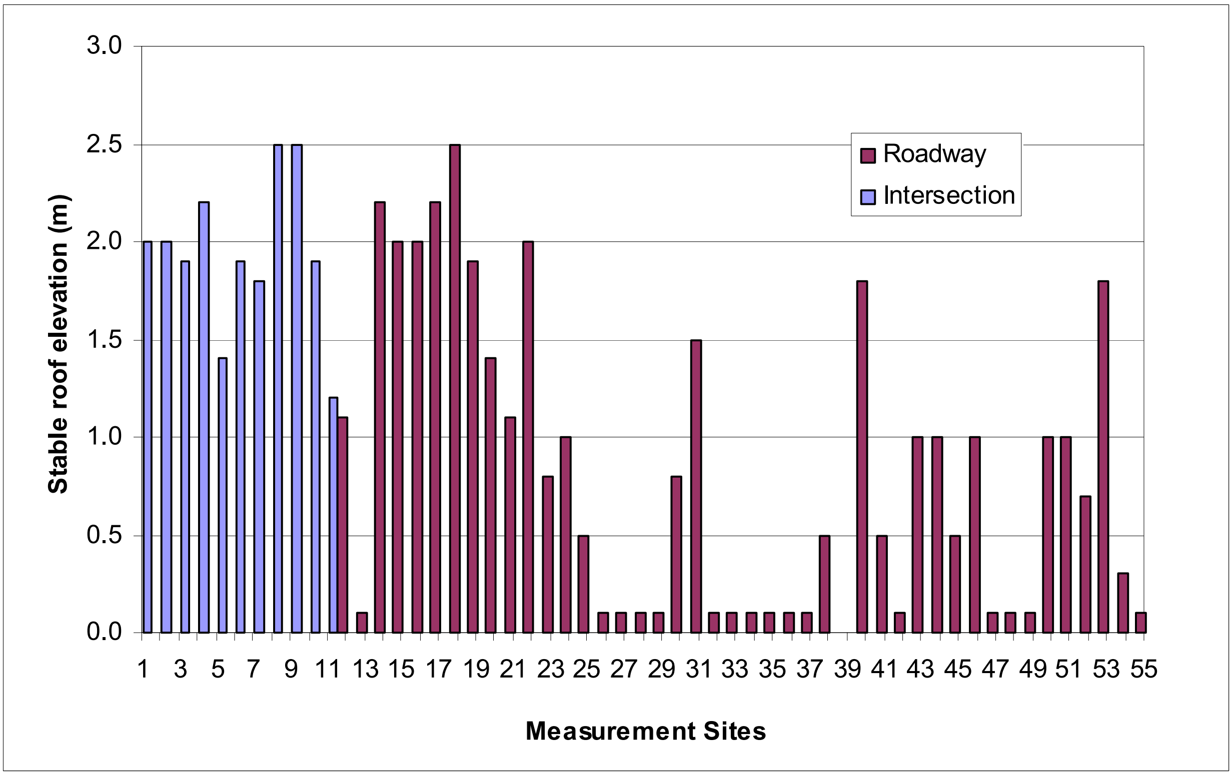


*Figure 7-3 Measured height of roof-softening in intersections and roadways in South African collieries*

***Figure 7-4 An example of roof-softening in a coal mine in the USA (courtesy of Dr. C. Mark)***

The height of instability measurements are also compared against the investigations conducted on falls of ground fatalities for the period 1970 to 2003 by Vervoort (1990) and as part of this thesis.

Vervoort (1990) investigated the falls of ground fatalities in South African collieries for the period 1970-1988. A similar study has also been conducted as part of this thesis covering the period 1989 to 2003. Figure 7-5 compares the two data sets with respect to thickness of fall. This Figure indicates that, for 33 year period, a large proportion of fall of ground accidents was due to relatively small falls of ground. However, the proportion of larger falls of ground has increased slightly in the recent data.

The cumulative distribution of thicknesses which caused FOG fatalities during the period 1989 to 2003 and the roof-fracturing heights measured underground are shown in Figure 7-6.

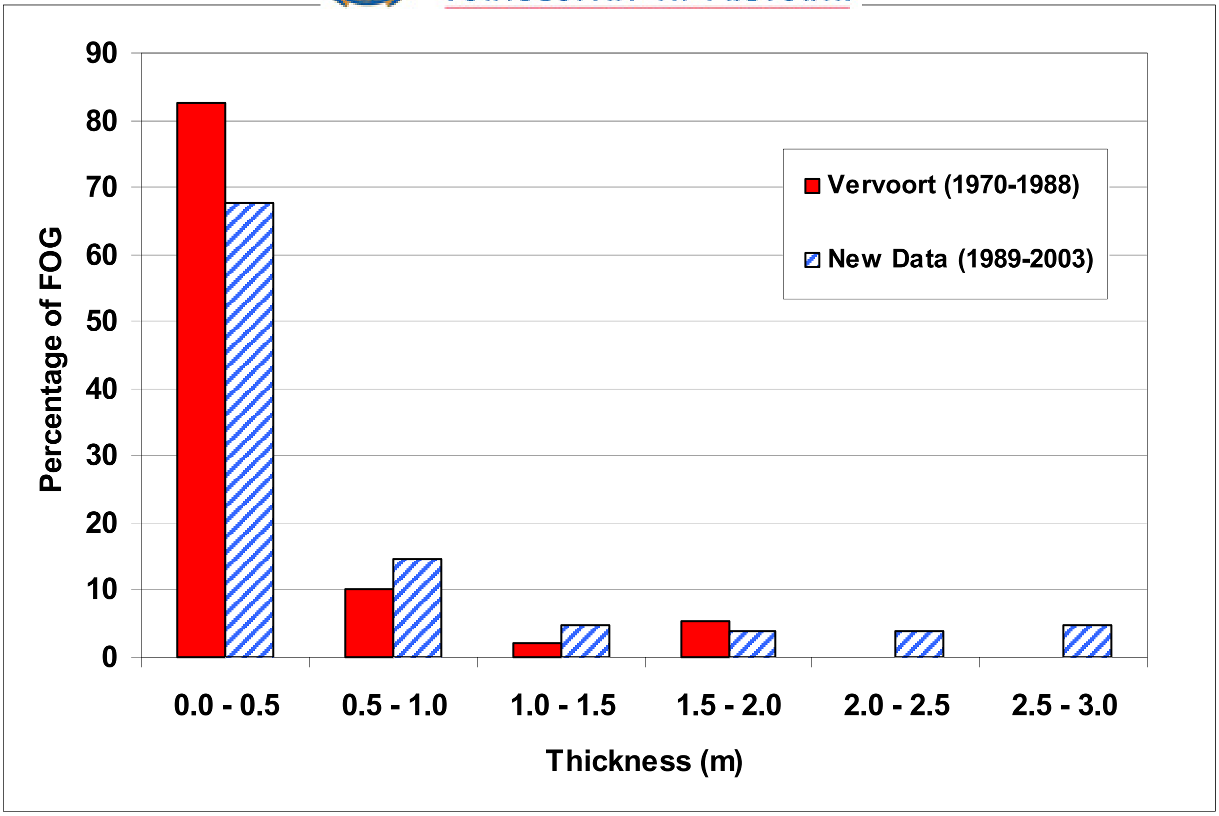


***Figure 7-5 The vertical dimension (thickness) of FOG causing fatalities for the period 1970 – 1995***

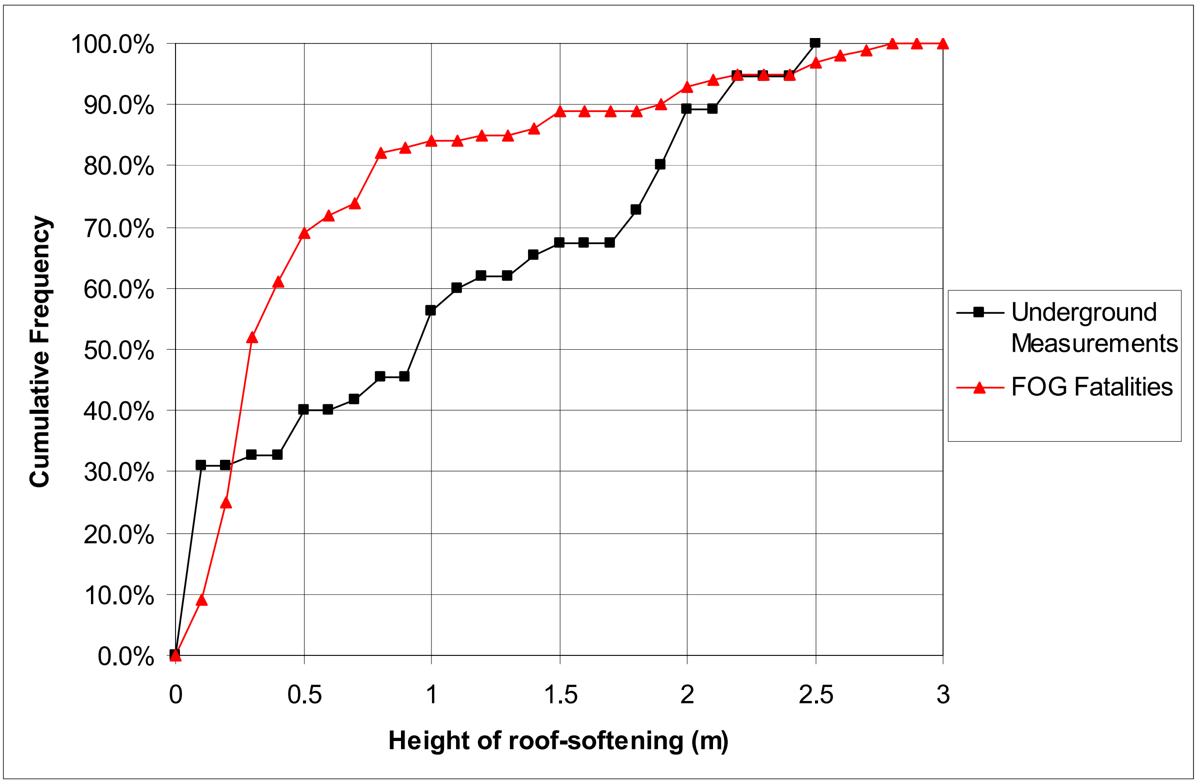


***Figure 7-6 Cumulative distribution of FOG thicknesses and the height of roof softening measured underground***

Using the underground measurement data, a comparison was also made between the magnitude of deformations in intersections and roadways. The results indicated that, for a 41 per cent increase in the span (taken across the diagonal of an intersection) relative to the roadway span, the magnitude of the displacement in the roof increased by a factor of about four on average, Figure 7-7.

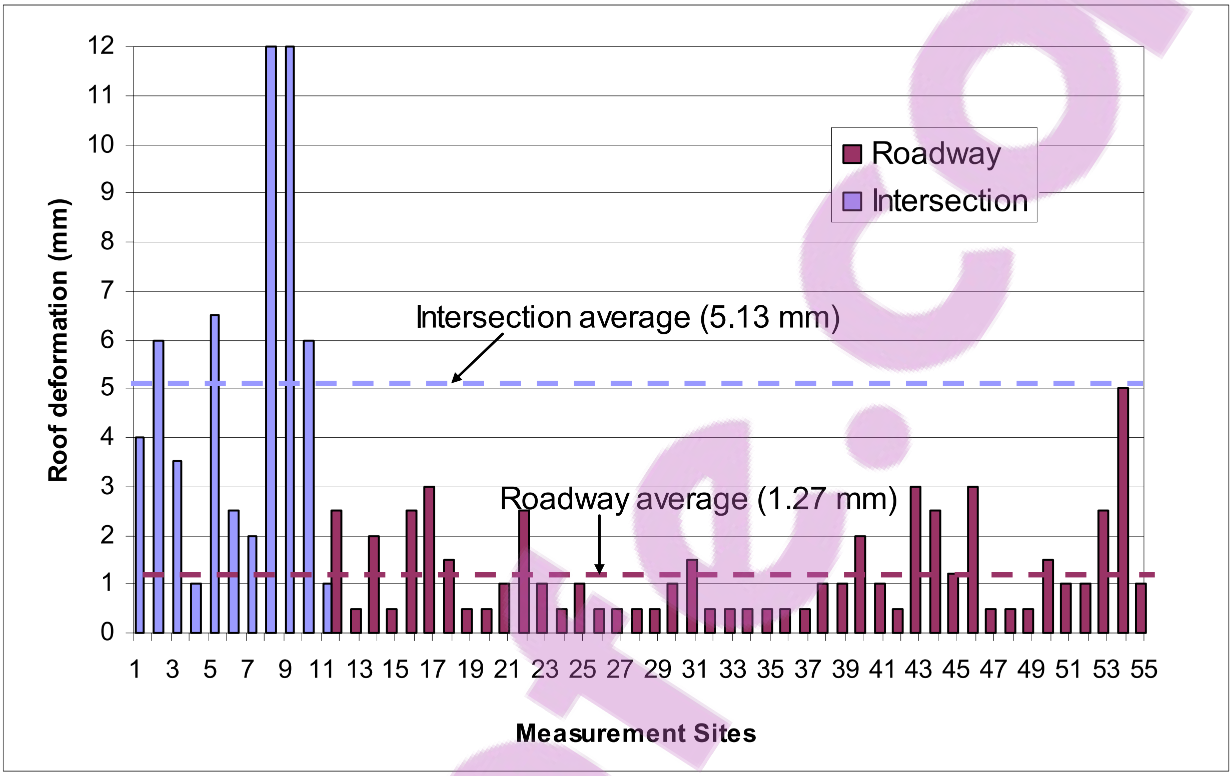


***Figure 7-7 Measured deformations in intersections and roadways***

The magnitude of measured deformations is also evaluated against the maximum theoretical deflection in a built-in beam using the following formula:

$$\eta_{\max} = \frac{\rho g L^4}{32 E t^2} \qquad [7\text{-}9]$$

where $L$ = roof span (width of roadway)

$t$ = thickness of roof layer (m)

$\rho$ = density of suspended strata (kg/m$^3$)

$g$ = gravitational acceleration (m/sec$^2$)

$E$ = Elastic modulus (Pa)

If the roof span ($L$) in the above formula increases by 41 per cent due to the diagonal width of the intersections, the deformation increases by a factor of 4.0. This is in accord with the findings in Figure 7-7.

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The results obtained from the magnitudes of deformations in intersections and roadways reveal that there is a significant correlation between the underground measurements and the beam theory. Also, in the light of the similar correlations found in other Chapters, it is therefore concluded that the roof behaviour in South African collieries can be classified as similar to that of a clamped beam.

The results also suggest that based on the height of softening measurements and the fall of ground fatality data collected over for 33 years, the suspension and beam building mechanisms (with improvements as discussed further in this Chapter) that have been used in South African collieries for many years are, in general, applicable where the appropriate conditions exist. It is however essential to determine the correct support mechanism to ensure the stability of roof.

From the results presented above, the roof behaviour model presented in Figure 7-8 is suggested.

This model suggests that when an underground opening is made, the portion of the strata directly above the opening loses its original support and the stress equilibrium is disturbed. The roof starts to sag under the gravitational and/or horizontal forces (irrespectively) up to a height where there is a competent layer and a new equilibrium is reached. In the case of absence of competent layers, as the lower layers start losing their integrity, the height of instabilities increase further into the roof. To maintain the stability, it is essential to keep the immediate, softened zone stable (Figure 7-8) using either suspension or the beam building mechanism. In beam building mechanism, roof bolts in this zone force all the bolted layers to sag with the same magnitude; the layers within the bolting range thus act like a solid beam supporting the bolted horizon as well as the surcharge load due to softened layers higher into the roof.

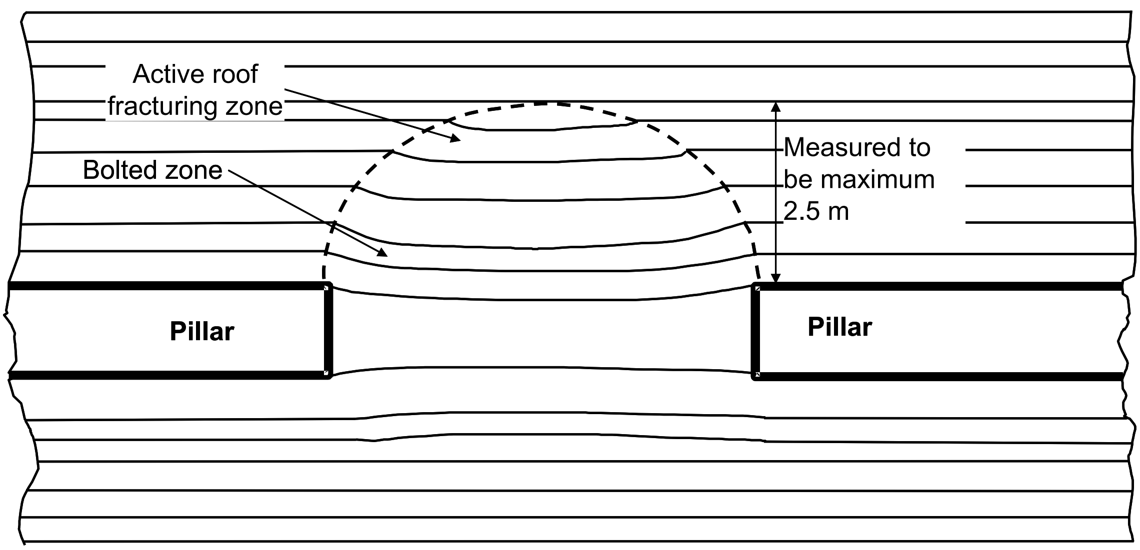


***Figure 7-8      Zone of roof softening***

### 7.3.1 Failure and support mechanisms

As indicated in the above model, before a roof bolt system is designed for a certain support mechanism, it is important to establish the geology for at least 2.5 m into the roof (based on measurements), which will assist in identifying the expected roof behaviour and in determining the support mechanism to be used.

If the immediate roof is very weak, but a competent layer exits higher in the roof, the suspension support mechanism is indicated. However, when the entire roof consists of a succession of thin beams, none of which are self-supporting, the suspension principle cannot be applied in this case beam building mechanism is suggested.

It is suggested that before any decision has been made regarding the support system, a detailed geotechnical investigation should be conducted (especially in greenfield studies) to determine the heights of roof softening, which can be assumed to be extended up to the "poor" quality layers. This investigation can be carried out using the standard laboratory tests, impact splitting tests, RQD or Rock Mass Rating.

In the suspension mechanism, the lower (loose) layer is suspended from the upper (competent) layer using roof bolts (van der Merwe and Madden, 2002), Figure 2-12. This creates a surcharge load and increases the maximum tensile stress in the upper layer, above the abutments. This surcharged tensile stress ($\sigma_{xx(\max)}$ in Pa) can be calculated using the following formula;

$$\sigma_{xx(\max)} = \frac{\rho g (t_{com} + t_{lam}) L^2}{2 t_{com}^2} \qquad [7\text{-}10]$$

where, $\rho$ = density of suspended strata (kg/m$^3$)

$g$ = gravitational acceleration (m/s$^2$)

$L$ = span (bord width or intersectional diagonal width) (m)

$t_{com}$ = competent layer thickness (m)

$t_{lam}$ = laminated lower strata thickness (m)

For stability to take place, the tensile strength of the competent layer should be greater than the tensile stress generated in this layer due to surcharge load.

It should be noted that as mentioned above, the thickness of competent layer, the position of competent layer, the bord widths, the thickness of suspended strata and the strength of competent layer will vary in nature. It is therefore suggested in determination of the applicability

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of the suspension mechanism using Equation [7-10] that a minimum of probability of stability of (PoS) 99 per cent should be attained.

Regarding the tensile strength of rock mentioned above, it should be noted that the tensile strength of rock is determined by the resistance of rock to tension. The failure of rock under tension is invariably abrupt with total loss of cohesion and load carrying ability. Direct determination of tensile strength for rock, i.e. "pull tests", is difficult, mainly because of involved specimen preparation. Indirect methods are most commonly used for determining the tensile strength.

The Brazilian (disc) method has proven to be a useful technique for a wide range of rock materials. It has, however, been found that the tensile strength determined by Brazilian tests is usually higher than the direct pull test value.

In general, while a rock material may have a tensile strength, a rock mass is often assumed to have very low tensile strength. This assumption is considered appropriate given the existence of joints and other defects in the rock mass. It is suggested that a detailed analysis should be conducted in determining the tensile strength of coal measure rock.

## 7.4 Roof bolting mechanisms

### 7.4.1 Suspension mechanism

As mentioned in Chapter 2.0, suspension mechanism (Figure 2-12) is the most easily understood roof bolting mechanism. While the majority of roof bolts used are resin point anchors, mechanical anchors are also uncommonly used (2 per cent only, Henson, 2005).

The design of roof bolt systems based on the suspension principle has to satisfy the following requirements:

- The strength of the roof bolts has to be greater than the relative weight of the loose roof layer that has to be carried.
- The anchorage forces of the roof bolts have to be greater than the weight of the loose roof layer.

The safety factor ($SF_{sus}$) of a bolting system in suspension mechanism is given by:

$$SF_{sus} = \frac{nP_f}{\rho g t_{lam}} \quad [7\text{-}11]$$

where, $\rho$ = density of suspended strata (kg/m$^3$)

$g$ = gravitational acceleration (m/sn$^2$)

$P_f$ = resistance of bolting system calculated from SEPT (kN)

$t_{lam}$ = thickness of loose layer or layers (m)

$n$ = number of bolts/m$^2$

$n$ can be calculated as follows:

$$n = \frac{k}{Ld} \quad [7\text{-}12]$$

where $d$ = distance between the rows of roof bolts (m)

$L$ = span (bord width) (m)

$k$ = number of bolts in a row

### 7.4.2 Beam building mechanism

Classical beam theory was first used by Obert and Duvall (1967) in the design of roof bolt patterns. However, the derivations in this chapter are taken directly from a standard reference (Popov, 1978) to establish an improved design methodology for the beam building mechanism, which takes into account, where appropriate, the surcharge load (assumed to be parabolic) generated by the softened section above the bolted horizon. This phenomenon has been ignored in the design of roof support systems since 1970s by the introduction of beam building mechanism in South Africa.

The first consideration in the design of beam building mechanism is to determine the minimum required thickness of the beam which will be stable from the tensile failure point of view.

The maximum tensile stress must be smaller than the tensile strength of upper layer of built beam with an appropriate PoS (99 per cent). The maximum tensile stress in a built-beam with a parabolic surcharge load can be calculated as:

$$\sigma_{xx}\left(\pm\frac{L}{2}\right) = \frac{2}{5}\frac{\rho g L^2}{h^2}(h + h_1) \quad [7\text{-}13]$$

The tensile stress in the lower surface at mid-span of built-beam is:

$$\sigma_{xx}(0) = \frac{9}{40}\frac{\rho g L^2}{h^2}(h + h_1) \quad [7\text{-}14]$$

Beams are subjected to transverse loads which generate both bending moments $M(x)$ and shear forces $V(x)$ along the beam. The bending moments cause horizontal stresses, $\sigma_{xx}$, to arise through the depth of the beam, and the shear forces cause transverse shear-stress distributions $\tau_{xy} = \tau_{yx}$ through the beam cross section as shown in Figure 7-9.

An important consideration in beam theory is that the top and bottom surfaces of the beam are free of shear stress, and the shear stress distribution across the beam is parabolic. As a consequence of this, the maximum shear stress (at the neutral axis of the beam) is given by:

$$\tau_{\max}(x) = \frac{3V(x)}{2A} \qquad [7\text{-}15]$$

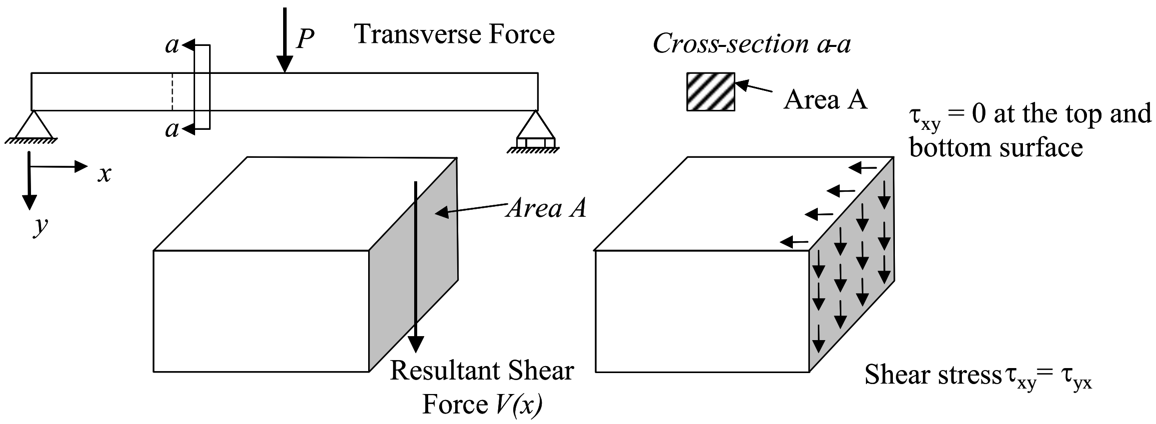


***Figure 7-9 Beam with transverse shear force showing the transverse shear stress developed by it***

The shear force distribution $V(x)$ is zero at the centre of a symmetrically loaded beam, and rises to a maximum at the end where it equals ½ of the total load. If the composite beam thickness is taken to be equal to the bolt length $h$, and the surcharge is parabolically distributed with a maximum height $h + h_1$ (Figure 7-8 and Figure 7-10), then

$$V_{\max} = \frac{1}{3}\rho g\left(h + h_1\right)L \qquad [7\text{-}16]$$

And from Equation [7-15]:

$$\tau_{\max} = \frac{1}{2}\frac{\rho g}{h}\left(h + h_1\right)L \qquad [7\text{-}17]$$

Where $h$ = built beam thickness (m)

$h_1$ = additional surcharge thickness (m)

$L$ = span (m)

$\rho$ = density of strata (kg/m$^3$)

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$g$ = gravitational acceleration (m/s$^2$)

For the built composite beam to act as a single entity, the shear stress given by Equation [7-17] has to be overcome by the action of the bolts. Two types of resistance are provided: frictional due to bolt pre-tensioning, and intrinsic shear strength of the bolts.

Neglecting the inter-layer cohesion and layer deadweight, the frictional shear resistance of tensioned roof bolts can be calculated using the following well-known formula (Wagner, 1985):

$$T_R = nF_p\mu \qquad [7\text{-}18]$$

where $n$ is number of bolts per square meter, $F_p$ is the pre-tension of bolt (usually 50 kN), and $\mu$ is the coefficient of friction between the layers.

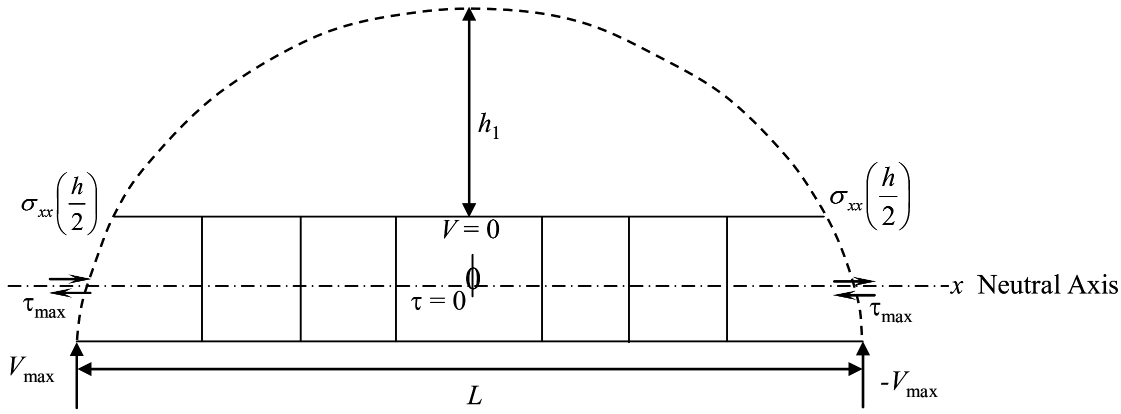


***Figure 7-10 Computation and distribution of shear stress in a beam***

In order to determine the coefficient of friction between the layers, a number of borehole samples from 5 collieries were obtained. All samples (61 mm in diameter and approximately 300 mm in length) were collected from the expoloration drilling and wraped in plastic bags to avoid weathering. To ensure for the failure to take place at the contacts between the different rock types, the top and bottom parts of the samples were cut and placed in a shear box. As shown in Table 7-5, the rock types and the contact conditions varied widely.

Despite the variation in rock and contact types, the standard deviation of the friction angle is relatively low: 9.2 per cent of the average. Note that the samples as tested may have been influenced by the drilling process. The influence of this has not been determined.

The shear strength of bolts also generates shear resistance, which must be considered in the design. This can be calculated using the following formula:

$$T_B = nS_R \qquad [7\text{-}19]$$

where $S_R$ is shear strength of a bolt (in kN).

***Table 7-5 Results of shear box tests on various contacts typically found in coal mines***

| Number | Contact details | Friction angle (deg.) | Coefficient of friction |
|---|---|---|---|
| 1 | coal/sandstone | 23.6 | 0.44 |
| 2 | shale/sandstone | 24.3 | 0.45 |
| 3 | coal/shale | 24.8 | 0.46 |
| 4 | shale/sandstone | 21.7 | 0.40 |
| 5 | shale/sandstone | 24.7 | 0.46 |
| 6 | shale/sandstone | 29.8 | 0.57 |
| 7 | coal/sandstone | 25.8 | 0.48 |
| 8 | coal/sandstone | 25.8 | 0.48 |
| 9 | sandstone/carbonaceous sandstone | 24.3 | 0.45 |
| 10 | coal/shale | 22.9 | 0.42 |
| 11 | sandstone/carbonaceous shale | 25.1 | 0.47 |
| 12 | coal/carbonaceous shale | 23 | 0.42 |
| 13 | sandstone/carbonaceous shale | 20.2 | 0.37 |
| 14 | coal/coal | 27.8 | 0.53 |
| 15 | coal/calcite | 26.8 | 0.51 |
| 16 | sandstone/carbonaceous shale | 22.7 | 0.42 |
| 17 | coal/sandstone | 27.7 | 0.53 |
| 18 | coal/sandstone | 25.1 | 0.47 |
| 19 | coal/laminated sandstone | 25.2 | 0.47 |
| **Average** | | **24.8** | **0.46** |
| **Standard deviation** | | **2.3** | **0.05** |
| **Standard deviation as a percentage of average** | | **9.2** | **10.4** |

There have been extensive studies in the past to determine the shear strength of a bolt. In South Africa, it has previously been accepted that 50 per cent of the *ultimate* tensile strength (UTS) of a bolt is approximately equal to the shear strength of a bolt (Wagner, 1985). However, Azuar (1977) concluded, from tests of resin-grouted bolts embedded in concrete, that the shear resistance of a joint when the bolt is installed perpendicular to the joint, is about of 90 per cent of the UTS. Roberts (1995) reported shear test results for smooth bars, rebars and cone bolts. He compared results of shearing at two interfaces (double shear) to a single interface shear and found that the former was not simply double the latter, as true symmetry did not exist in the case of double shear. Shear failure would occur at one interface first and subsequently resulted in failure of the other interface. From tests, he noted that a 16 mm diameter rebar had a static shear strength of approximately 90 per cent of the UTS. Canbulat et al. (2006), based on laboratory shear tests, also concluded that the shear strength of full-column roof bolts that are currently being used in South Africa is approximately 87 per cent of the ultimate tensile strength with very consistent results. Since this simple assumption will determine the required bolt length and density, it is suggested that the shear strength of a full column bolt is taken to be equal to

90 per cent of the UTS of a bolt (based on 600 MPa for standard roof bolts in South African collieries e.g. 190 kN for 20 mm bolts).

Equation [7-19] then becomes:

$$T_B = 0.9nS_B \tag{7-20}$$

where $S_B$ is the ultimate tensile strength of a bolt (in kN).

The shear resistance of a bolting system can therefore be determined as follows:

$$T_{TOTAL} = n(F_p\mu + 0.8S_B) \tag{7-21}$$

And for stability this has to exceed the value given by Equation [7-17].

$$SF = \frac{T_{TOTAL}}{\tau_{\max}} \tag{7-22}$$

Another important consideration in beam building mechanism occurs when the roof softening height is within the bolted horizon (Figure 7-11). This usually occurs when the bolts are installed late and the separation has already taken place and destroyed the cohesion between the layers or under excessive stress conditions.

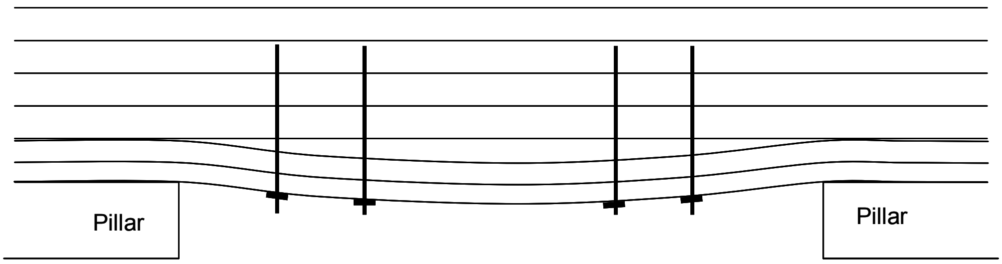


***Figure 7-11 Bed separation within the bolted horizon***

In this case, safety factor ($SF_{slide}$) of resistance to sliding of the bolting system should be calculated using the bond strength ($B_s$) between the resin, rock and the bolt using the following formula:

$$SF_{slide} = \frac{kB_S l_{cap}}{Lt_{loose}d\rho g} \tag{7-23}$$

Where $B_S$ = Bond strength or grip factor (kN/mm)

$d$ = distance between the rows of roof bolts (m)

$L$ = span (bord width) (m)

$t_{loose}$ = thickness of separated layer (m)

$k$ = number of bolts in a row

$l_{cap}$ = capsulation length (bolt length – $t_{loose}$) (m)

$\rho$ = density of strata (kg/m$^3$)

$g$ = gravitational acceleration (m/s$^2$)

Bond strength is measured through short encapsulation pull tests (SEPT). In order to measure the bond strength, it is necessary to shear the bond on the bolt-resin or resin-rock interface. With the modern high-strength, high-stiffness, polyester resins, it has been found that a bond length of 250 mm is appropriate for determining the bond strength.

Bond strength ($B_S$) is defined as:

$$B_S = \frac{Maximum\,Load\;Achieved\,(kN)}{Encapsulation\,Length\,(mm)} \quad [7\text{-}24]$$

Similar to suspension mechanism, to avoid the failure of roof bolts in tension, the safety factor ($SF_{bolt}$) of roof bolts should also be determined. The following formula can be used to calculate the safety factor of roof bolts:

$$SF_{slide} = \frac{kP_{bolt}}{Lt_{loose}d\rho g} \quad [7\text{-}25]$$

Where $P_{bolt}$ = bolt yield strength (kN)

$d$ = distance between the rows of roof bolts (m)

$L$ = span (bord width) (m)

$t_{loose}$ = thickness of separated layer (m)

$k$ = number of bolts in a row

$\rho$ = density of strata (kg/m$^3$)

$g$ = gravitational acceleration (m/s$^2$)

## 7.5 Determination of stability of the immediate layer between the roof bolts

In the case of thin roof beds the spacing between bolts is critical. Wagner (1985) suggested that the distance between the bolts should not exceed a value of 10 times the thickness of the layer. However, to prevent the failure of the immediate roof between the bolts, the tensile stress between the bolts for the immediate layer may be calculated by assuming that the bolts create a fixed beam between them. If the tensile stress between the bolts exceeds the tensile strength of the material then the distance between the bolts should be reduced or an areal coverage system should be used. The safety factor of roof between the bolts may be calculated again from the clamped beam equation (van der Merwe and Madden, 2002):

$$SF = \frac{2t_{imm}^2\sigma_x}{\rho g t_{imm} L_b^2} \quad [7\text{-}26]$$

where, $\sigma_x$ = tensile strength of immediate roof (MPa)

$\rho$ = density of immediate layer (kg/m$^3$)

$g$ = gravitational acceleration (m/s$^2$)

$L_b$ = distance between the bolts (m)

$t_{imm}$ = thickness of immediate layer (m)

Note that in the case of low modulus layers overlaying the immediate layer, surcharge loading should be taken into account by suitably increasing $t$ in the numerator of Equation [7-26].

In the case of failure of very thin layers (<100 mm) between the roof bolts, it is certainly impossible to prevent the failure using only roof bolts, in this case, if the layer cannot be mined out due to contamination concerns, areal coverage in the form of wire-mesh, W-straps and/or shotcrete is recommended.

## 7.6 Probability density functions of design parameters and random selection

As indicated in Section 7.2 that the fundamental to Monte Carlo method is the process of explicitly representing the uncertainties by specifying inputs as probability distributions. Probability density functions are the tools used to estimate the likelihood that random variable values will occur within certain ranges. There are two types of random variables, namely discrete and continuous. A discrete (finite) random variable can take only a countable number of distinct values. A continuous (infinite) random variable can however takes an unknown number of possible samples and the samples are not countable, but are taken from a continuous interval. Because few, if any, geotechnical properties will behave as a discrete probability space, discrete distributions are not presented herein.

The probability density function is a function that assigns a probability to every interval of the outcome set for continuous random variables. The probability density function is denoted $f_x(x)$, where $x$ is the random variable itself and $x$ is the value that the continuous random variable can take on. Probability functions have the following properties (Jones et al., 2002):

1. The function is always nonnegative, $f_x(x) \geq 0$
2. The area under the function is equal to one, $\int_{-\infty}^{\infty} f_x(x)dx = 1$

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3. The probability that a random value, $X$, from the distribution is between $a$ and $b$ is $P[a \le x \le b] = \int_a^b f_x(x)dx$

Cumulative probability distribution (CDF) functions have the value at $x_0$ corresponding to the probability that a random value, $X$, from the distribution will be less than or equal to $x_0$.

For a continuous distribution, this can be expressed mathematically as $\Pr(X \le x_0 = \int_{-\infty}^{x_0} f(x)\,dx$

Over 25 special continuous probability density distributions exist. The following 10 most commonly used distributions are however utilised in this thesis:

1. Beta
2. Erlang
3. Exponential
4. Gamma
5. Logistic
6. Lognormal
7. Normal
8. Pert
9. Weibull

Rather than focus on the derivations, useful properties of these distributions are presented in Table 7-6.

In order to determine the best fit probability density distributions for each of the input parameters used in the design, the underground measurement data collected throughout this study has been analysed using the Anderson-Darling goodness of fit test.

***Table 7-6 Summary of probability distributions (after EasyFit user manual, 2006)***

| Distribution | Parameters | Density distribution function | Cumulative distribution function | Definitions |
|---|---|---|---|---|
| Beta | $\alpha_1$ continuous shape<br>$\alpha_2$ continuous shape | $f(x)=\dfrac{x^{\alpha_1-1}(1-x)^{\alpha_2-1}}{B(\alpha_1,\alpha_2)}$ | $F(x)=\dfrac{B_x(\alpha_1,\alpha_2)}{B_x(\alpha_1,\alpha_2)}\equiv I_x(\alpha_1,\alpha_2)$ | $B$ = Beta function<br>$B_x$ = Incomplete beta function |
| Erlang | $m$ integral shape<br>$\beta$ continuous scale | $f(x)=\dfrac{1}{\beta(m-1)!}\left(\dfrac{x}{\beta}\right)^{m-1}e^{-x/\beta}$ | $F(x)=\dfrac{\Gamma_{x/\beta}(m)}{\Gamma(m)}=1-e^{-x/\beta}\sum\dfrac{(x/\beta)^i}{i!}$ | $\Gamma$ = Gamma function<br>$\Gamma_x$ = Incomplete gamma function |
| Exponential | $\beta$ continuous scale | $f(x)=\dfrac{e^{-x/\beta}}{\beta}$ | $F(x)=1-e^{-x/\beta}$ | |
| Gamma | $\alpha$ continuous shape<br>$\beta$ continuous scale | $f(x)=\dfrac{1}{B\Gamma(x)}\left(\dfrac{x}{\beta}\right)^{\alpha-1}e^{-x/\beta}$ | $F(x)=\dfrac{\Gamma_{x/\beta}(\alpha)}{\Gamma(\alpha)}$ | $\Gamma$ = Gamma function<br>$\Gamma_x$ = Incomplete gamma function |
| Logistic | $\alpha$ continuous location<br>$\beta$ continuous scale | $f(x)=\dfrac{\sec h^2\left[\dfrac{1}{2}\left(\dfrac{x-\alpha}{\beta}\right)\right]}{4\beta}$ | $F(x)=\dfrac{1+\tan h\left[\dfrac{1}{2}\left(\dfrac{x-\alpha}{\beta}\right)\right]}{2}$ | *sec h* = hyperbolic Secant Function<br>*tan h* = hyperbolic Tangent Function |
| Lognormal | μ continuous<br>σ continuous | $f(x)=\dfrac{1}{x\sqrt{2\pi}\sigma'}e^{-\frac{1}{2}\left[\frac{\ln x-\mu'}{\sigma'}\right]^2}$ | $F(x)=\Phi\left(\dfrac{\ln x-\mu'}{\sigma'}\right)$ | $\mu'\equiv\ln\left[\dfrac{\mu^2}{\sqrt{\sigma^2+\mu^2}}\right]$<br>$\sigma'\equiv\sqrt{\ln\left[1+\left(\dfrac{\sigma}{\mu}\right)^2\right]}$<br>$\Phi(z)$ = Laplace-Gauss |

| Distribution | Parameters | Density distribution function | Cumulative distribution function | Definitions |
|---|---|---|---|---|
| | | | | integral |
| Normal | $\alpha$ continuous location<br>$\beta$ continuous scale | $f(x)=\frac{1}{\sqrt{2\pi\sigma'}}e^{-\frac{1}{2}\left[\frac{x-\mu}{\sigma}\right]^2}$ | $F(x)=\Phi\left(\frac{\ln x-\mu}{\sigma}\right)=\frac{1}{2}\left[erf\left(\frac{x-\mu}{\sqrt{2}\sigma}\right)+1\right]$ | $\Phi$ = Laplace-Gauss integral<br>$erf$ = error function |
| Pert | $\mu\equiv\frac{\min+4\,m..likely+\max}{6}$<br>$\alpha_1\equiv6\left[\frac{\mu-\min}{\max-\min}\right]$<br>$\alpha_2\equiv6\left[\frac{\max-\mu}{\max-\min}\right]$ | $f(x)=\frac{(x-\min)^{\alpha_1-1}(\max-x)^{\alpha_2-1}}{B(\alpha_1,\alpha_2)(\max-\min)^{\alpha_1+\alpha_2-1}}$ | $F(x)=\frac{B_z(\alpha_1,\alpha_2)}{B(\alpha_1,\alpha_2)}\equiv\mathrm{I}_z(\alpha_1,\alpha_2)$ | $\min$ = continuous boundary parameter ($\min<\max$)<br>$\mathrm{m.likely}$ = continuous parameter<br>($\min<\mathrm{m.likely}<\max$)<br>$\max$ = continuous parameter<br>$z\equiv\frac{x-\min}{\max-\min}$<br>$B$ = Beta function<br>$B_z$ = Incomplete beta function |
| Weibull | $\alpha$ continuous shape<br>$\beta$ continuous scale | $f(x)=\frac{\alpha x^{\alpha-1}}{\beta^\alpha}e^{-(x/\beta)^\alpha}$ | $F(x)=1-e^{-(x/\beta})^\alpha$ | |

### 7.6.1 Goodness of fit tests

There are several goodness of fit tests available, among them Kolmogorov-Smirnov (KS-test), Chi-square (CS-test) and Anderson-Darling (AD-test) goodness of fit tests.

Kolmogorov-Smirnov (Chakravart et al., 1967) test determines if two datasets differ significantly. An advantage of KS-test is that the distribution of the KS-test statistic itself does not depend on the underlying cumulative distribution function being tested. Another advantage is that unlike chi-square test, it is an exact test and does not require binned data an adequate sample size for the approximations to be valid. Despite these advantages, the KS-test has several important limitations:

1. It tends to be more sensitive near the centre of the distribution than at the tails.
2. The distribution must be fully specified. That is, if location, scale, and shape parameters are estimated from the data, the critical region of the KS-test is no longer valid. It typically must be determined by simulation.

The chi-square goodness of fit test (Snedecor and Cochran, 1989) is used to test if a sample of data came from a population with a specific distribution.

An important feature of the CS-test is that it can be applied to any distribution for which the CDF can be calculated. The chi-square goodness-of-fit test can only be applied to binned data (i.e., data put into classes) and the value of the chi-square test statistic is dependent on how the data is binned. Another disadvantage of the chi-square test is that it requires a sufficient sample size in order for the chi-square approximation to be valid. The test requires that the data first be grouped. The actual number of observations in each group is compared to the expected number of observations and the test statistic is calculated as a function of this difference.

Anderson-Darling test is a general test to compare the fit of an observed cumulative distribution function to an expected cumulative distribution function and can be applied to binned and unbinned data. AD-test is a modification of the KS-test and gives more sensitive to deviations in the tails of the distribution. The AD-test makes use of the specific distribution in calculating critical values. This has the advantage of allowing a more sensitive test and the disadvantage of that critical values must be calculated for each distribution.

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Since the the Anderson-Darling test implemented in EasyFit© (a computer program which determines the best fits based on goodness of fit tests) uses the same critical values for all the distributions and these values are calculated using the approximation formula, depending on the sample size, Anderson-Darling goodness of fit test and EasyFit© are utilised in this thesis to determine the best probability distributions representing the input parameters

The Anderson-Darling statistic ($A^2$) is defined as: (EasyFit© user manual, 2006)

$$A^2 = -n - \frac{1}{n}\sum_{i=1}^{n}(2i-1)\left[\ln F(X_i) + \ln(1-F(X_{n-1+i}))\right] \quad [7\text{-}27]$$

The hypothesis regarding the distributional form is rejected at the chosen significance level ($\alpha$) if the test statistic, $A^2$, is greater than the critical value.

### 7.6.2 Probability distributions of design parameters

Based on the load/strength models presented in Section 7.3 and 7.4, the following parameters' probability distributions will be determined to use in the probabilistic design of roof bolting systems:

- Bord width
- Distance between the bolts (in determining the roof bolt density)
- Pre-tension of roof bolts
- Height of roof softening
- Unit weight
- Bond strength
- Coefficient friction
- Bolt strength
- Tensile strength of rock
- Thickness of competent layer
- Thickness of suspended layer

Note that the distribution of roof bolt strength is calculated from the variation in the diameter of 18 mm roof bolts using a constant ultimate steel strength of 600 MPa.

A summary of the goodness of fit test results using the AD-test is summarised in Table 7-7.

***Table 7-7 Summary results of Anderson-Darling goodness of fit tests***

| **Parameter** | **Number of data points** | **Best fit probability distribution** | **Scale Parameter** | **Shape Parameter** | **Location parameter** |
|---|---|---|---|---|---|
| Bord width (m) | 258 | Logistic | 0.32 | N/A | 6.23 |
| Distance between the bolts (m) | 835 | Pert | 1.90 (mode) | 0.58 (min) | 3.31 (max) |
| Pre-tension of roof bolts (kN) | 122 | Pert | 29.80 (mode) | 18.92 (min) | 82.50 (max) |
| Height of roof softening (m) | 93 | Logistic | 0.17 | N/A | 0.65 |
| Unit weight ($MN/m^3$) | 168 | Erlang | 16.24 | 148.00 | N/A |
| Bond strength (kN/mm) | 46 | Lognormal | 0.29 | N/A | -0.87 |
| Coefficient friction ($^o$) | 19 | Lognormal | 0.10 | N/A | -0.78 |
| Bolt strength (kN) | 192 | Logistic | 0.36 | N/A | 120.40 |
| Tensile strength of sandstone (MPa) | 30 | Pert | 3.15 (mode) | 0.46 (min) | 5.19 (max) |
| Tensile strength of weak rock (MPa) | 66 | Pert | 0.79 (mode) | 0.32(min) | 3.44 (max) |
| Thickness of competent layer (m) | 43 | Weibull | 2.60 | 2.84 | N/A |
| Thickness of suspended layer (m) | 43 | Normal | 0.20 | N/A | 0.89 |

Note that as can be seen, the results presented in Table 7-7 are based on a limited number of data points. Therefore, certain best fit probability distributions obtained from Anderson-Darling goodness of fit tests are only marginally better than the others, such as Weibull distribution is only slightly better than the normal distribution for the thickness of the competent layer. This indicates that a more comprehensive database is required to establish the conclusive distributions.

## 7.7 Support design methodology

Using all above and the information presented in other Chapters of this thesis, the following step-by-step process is suggested in the design of roof support system:

1. Conduct a detailed geotechnical analysis to determine the height of roof softening. This can be achieved for existing operations from underground measurements and/or height of FOG, and for greenfield studies from the geotechnical rating systems (such as IST, CMRR and RQD). The details of these investigations can be found in Chapters 3, 4, 5, 6 and 7.

2. Determine the applicability of the suspension mechanism using Equation [7-10]. Note that a minimum PoS of 99 per cent is recommended to use the suspension mechanism with confidence.

3. Further detailed geotechnical analyses are required to determine the distributions of suspension and beam building mechanisms' input parameters. These input parameters and their probability distributions are summarised in Chapter 7.

4. Conduct short encapsulated pull tests to calculate the support resistance. Use the standard ISRM SEPT methodology.

5. For the appropriate support mechanism calculate the probability of stabilities of different length of roof bolts. Note that if required a sensitivity analysis into the distance between the rows of support elements, bord width, bond strength and pre-tension on roof bolts can be conducted at this stage. The details of this analysis can be found in Chapter 7.

6. Check the probability of stabilities achieved against the design criteria given in Table 7-4. If the design criteria is not achieved go back to Step 4.

7. If the design criteria is achieved in Step 7, check the stability between the roof bolts using Equation [7-26].

8. Determine the financial viability of the system. If the system is financially viable, implement it; otherwise conduct a detailed analysis into different bolting systems in Step 5.

9. Once the bolting system is implemented (i) monitor the support system and (ii) implement the appropriate quality control procedures using the methodology presented in Chapter 6.0.

10. As an on going procedure, use appropriate (developed for the specific conditions) section performance and risk rating system and continue monitoring the support system and the roof behaviour.

A design flow-chart summarising the above methodology is presented in Figure 7-12.

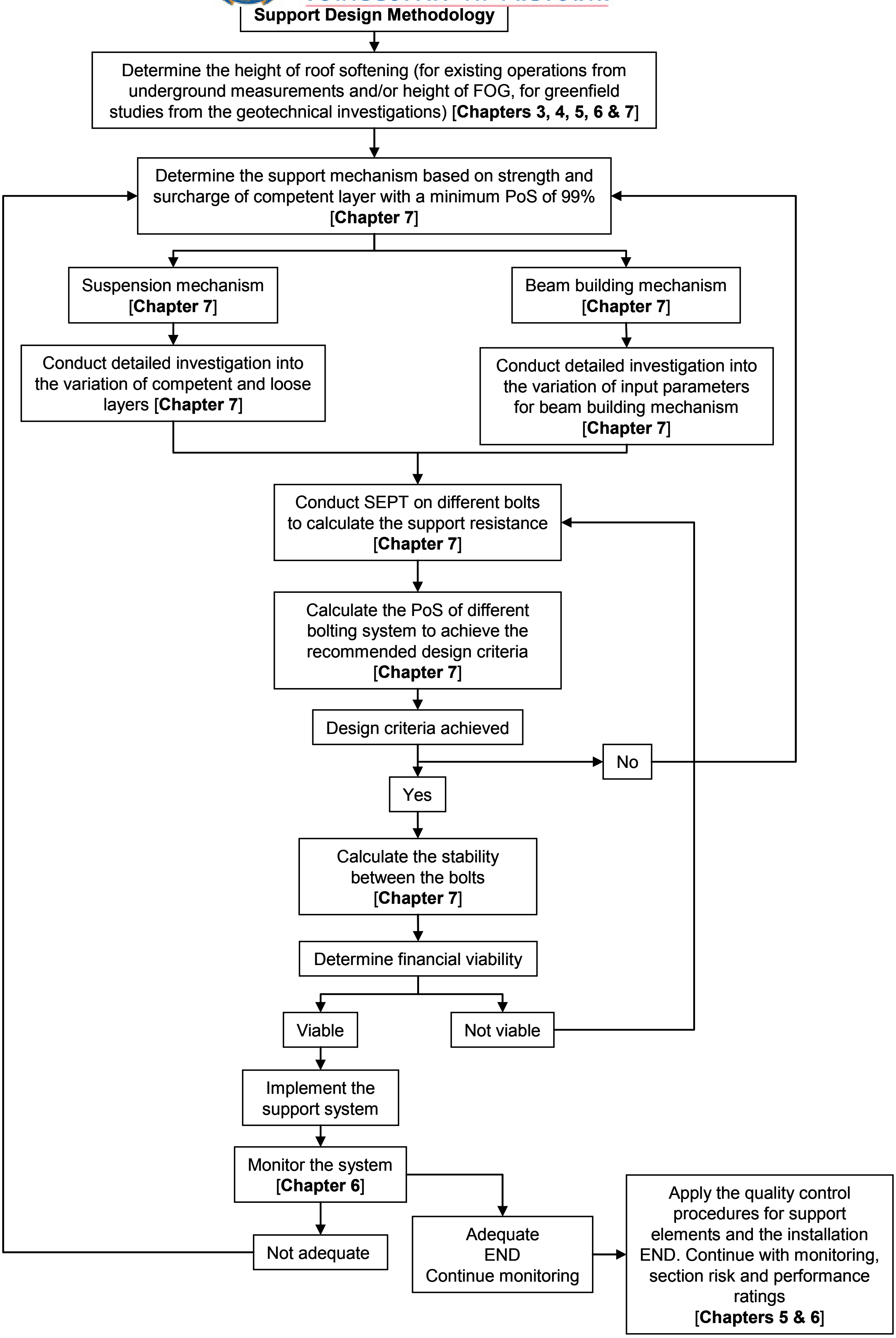


***Figure 7-12 Recommended support design methodology***

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### 7.8 Application of the probabilistic design approach to a case study

In the previous sections of this Chapter, a probabilistic design methodology is presented. In this section a verification of this design methodology will be demonstrated by applying it to a well-defined study with the aim of establishing the best support systems for a colliery in the Witbank Coalfield.

A detailed monitoring program was conducted in a bord and pillar section of Colliery 'A'. Using three sonic probe monitoring sites (two in roadways and one in an intersection) the roof behaviour was monitored and the height of roof softening data was obtained. Additional data was also obtained using a feeler-gauge (a telescopic pipe which contains a washer at the end and is inserted into a bolt hole to feel the bed separations). The mine experienced numerous roof falls for a period of time and an investigation into the thickness of roof falls was therefore conducted. This data was also combined with the sonic probe and fleer-gauge data to extend the height of roof softening database. Figure 7-13 summarises the data obtained from these three different techniques. It is evident from this Figure that the height of softening varies from 0.15 m to 1.65 m with an average of 0.65 m.

A detailed bord width measurement programme was also conducted and bord width offsets were measured in two different production sections. A frequency versus bord width graph is given in Figure 7-14. In these two sections, the bord widths were designed to be 6.5 m, but, in reality varied from 5.4 m to 7.6 m.

The immediate roof strata consisted of 0.1 to 1.0 m of coal, followed by a shale band approximately 0.3 m thick above which there is a further 3.0 m of coal. This data was obtained from 43 borehole logs that were available in the vicinity of the area where the bord width measurements and the height of roof softening data were collected. Figure 7-15 illustrates the distributions of thicknesses of the immediate and the upper roof coal layers. In this Figure, the immediate roof thicknesses included the skin coal and the shale band whilst the upper roof included the coal thickness overlain the immediate roof.

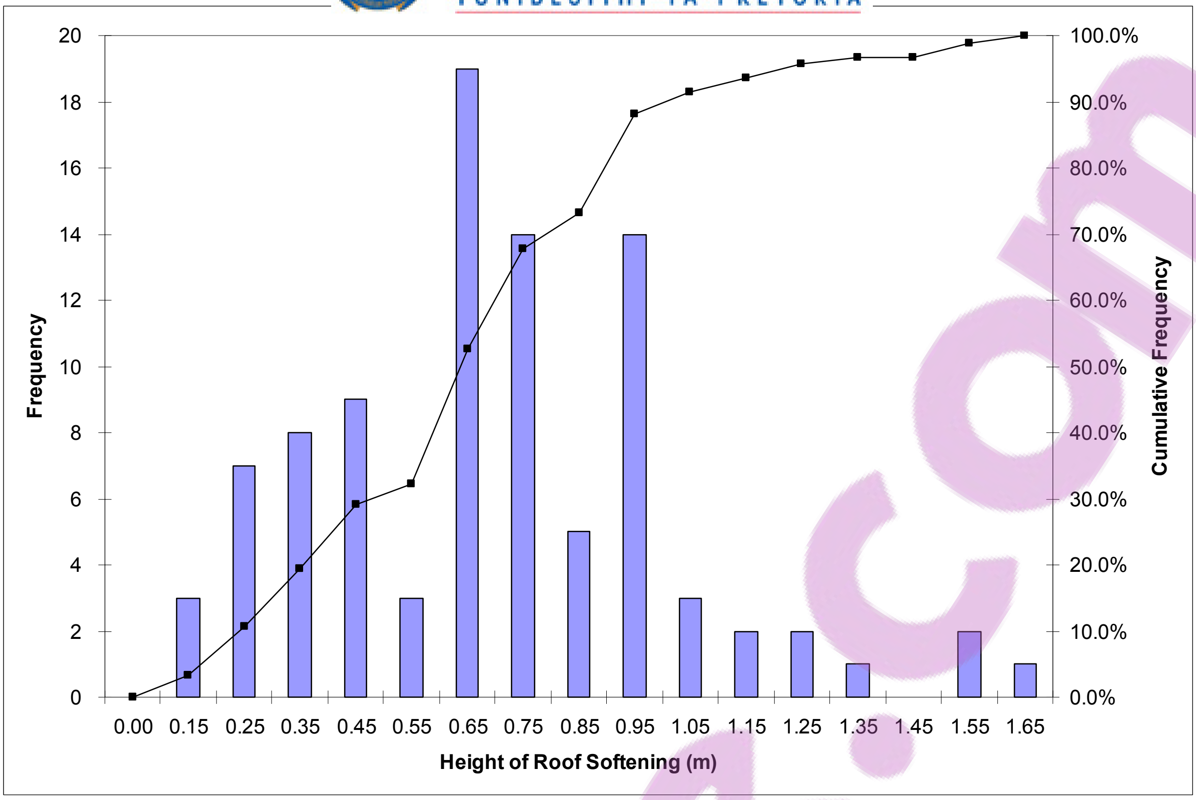


***Figure 7-13 Colliery "A" height of softening data obtained from the sonic probe extensometer results, feeler-gauge results and FOG data***

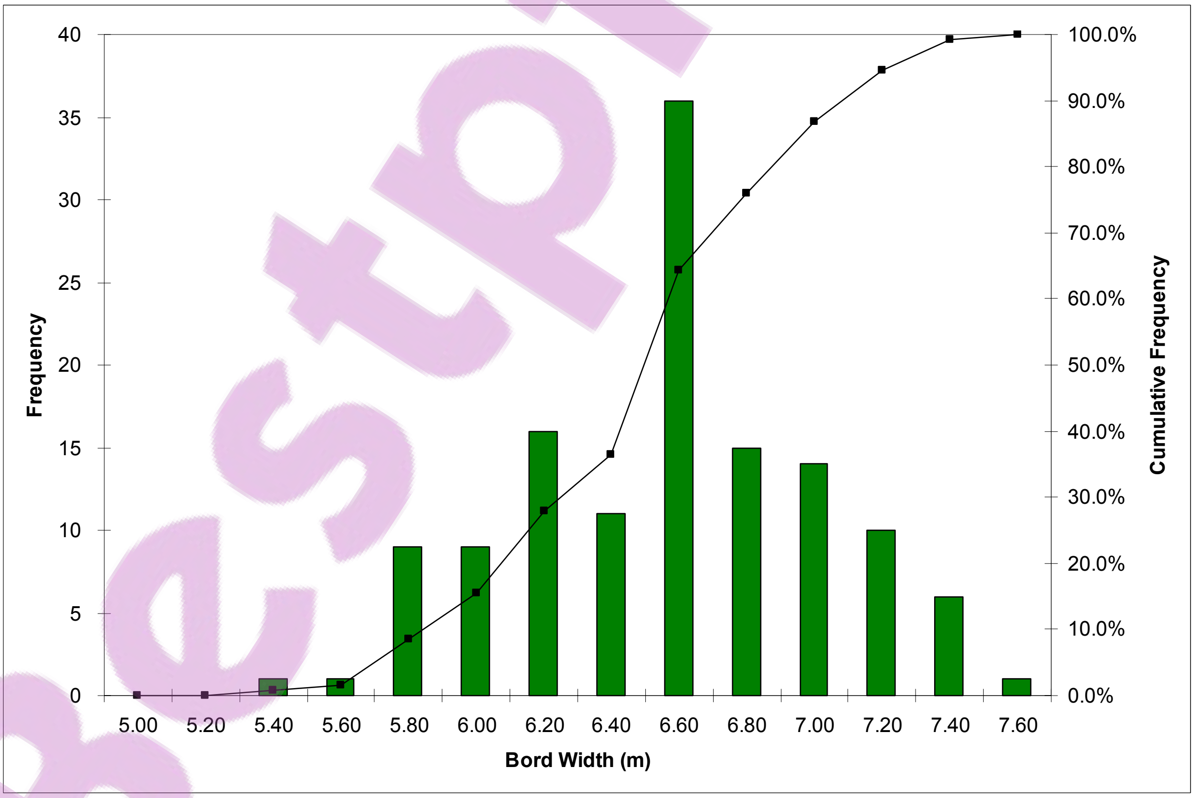


***Figure 7-14 Bord width distributions in the experiment site***

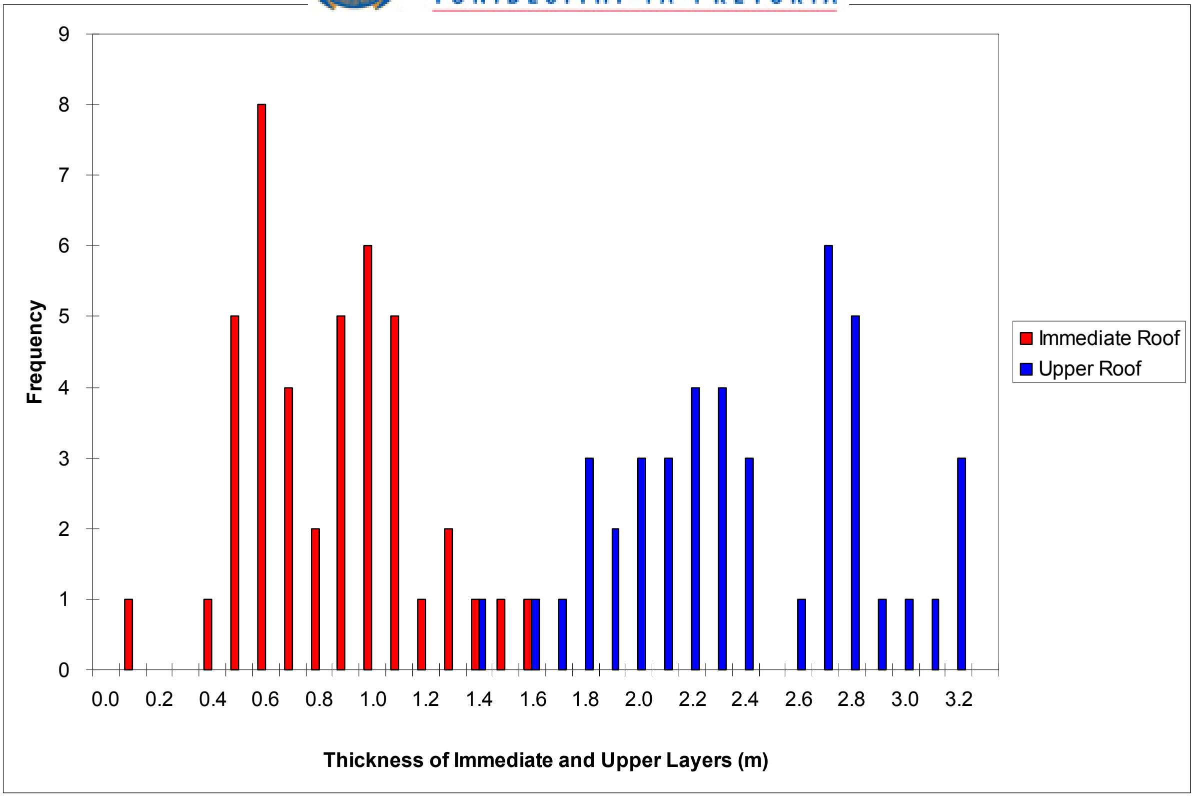


***Figure 7-15 Thickness of immediate and upper roof obtained from borehole logs***

A series of underground short encapsulation pull tests were carried out in near identical conditions in those two sections. Tests were performed using the 30 second spin and hold resin and 1.2 m long, 16 mm roof bolts, as currently being used by the mine. Note that due to the time laps between the tests and the need for the roofbolter in production schedule, tests were conducted in different areas of the sections.

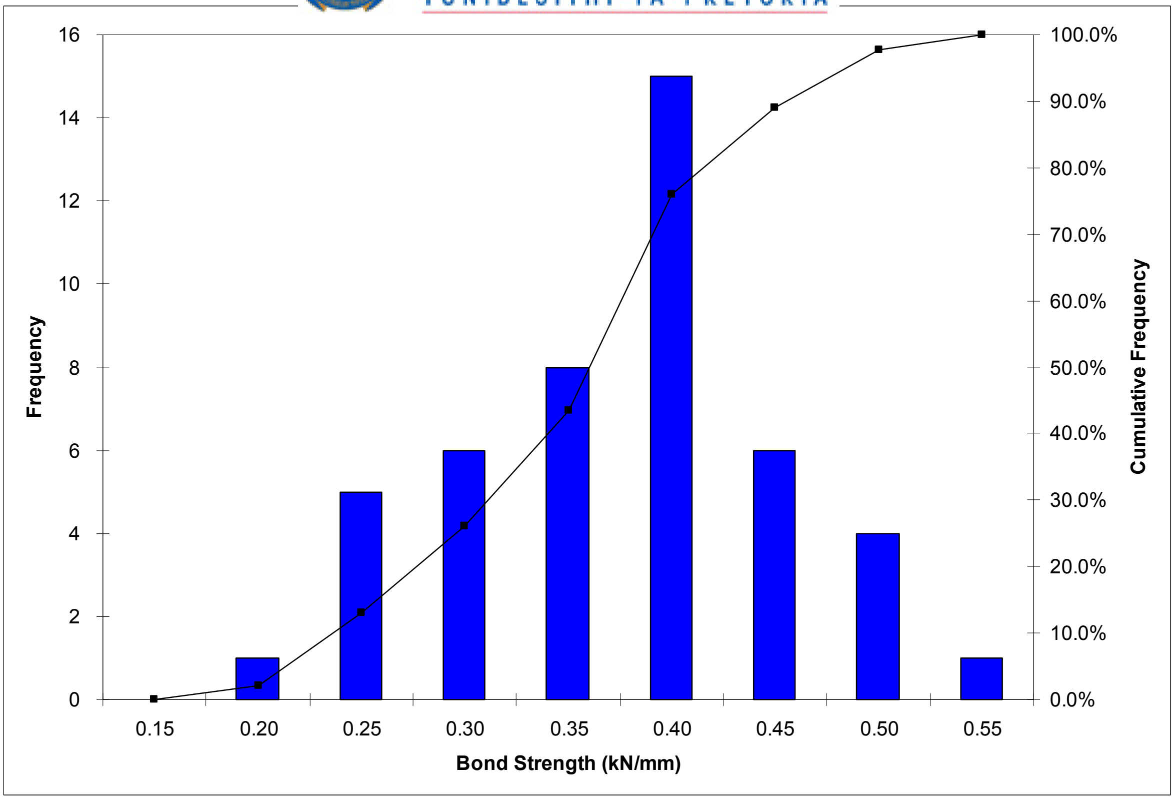


***Figure 7-16    Bond strength results obtained from SEPT in the experiment site***

In order to determine the tension on the roof bolts, over 145 roof bolts were tested using a torque-wrench. Figure 7-17 shows the distribution obtained from these measurements. As can be seen the tension on the roof bolts varied from 0 to 32.5 kN.

The distances between the rows of roof bolts were also measured in the monitoring site, Figure 7-18. Similar to bord widths, although the planned distance was 2.0 m, in reality it varied from 1.4 m to 3.2 m.

In order to determine the strength of roof bolts based on a constant 600 MPa ultimate steel strength, bolt diameter measurements were also taken over 80 bolts at the mine and the ultimate strength of roof bolts were determined, Figure 7-19.

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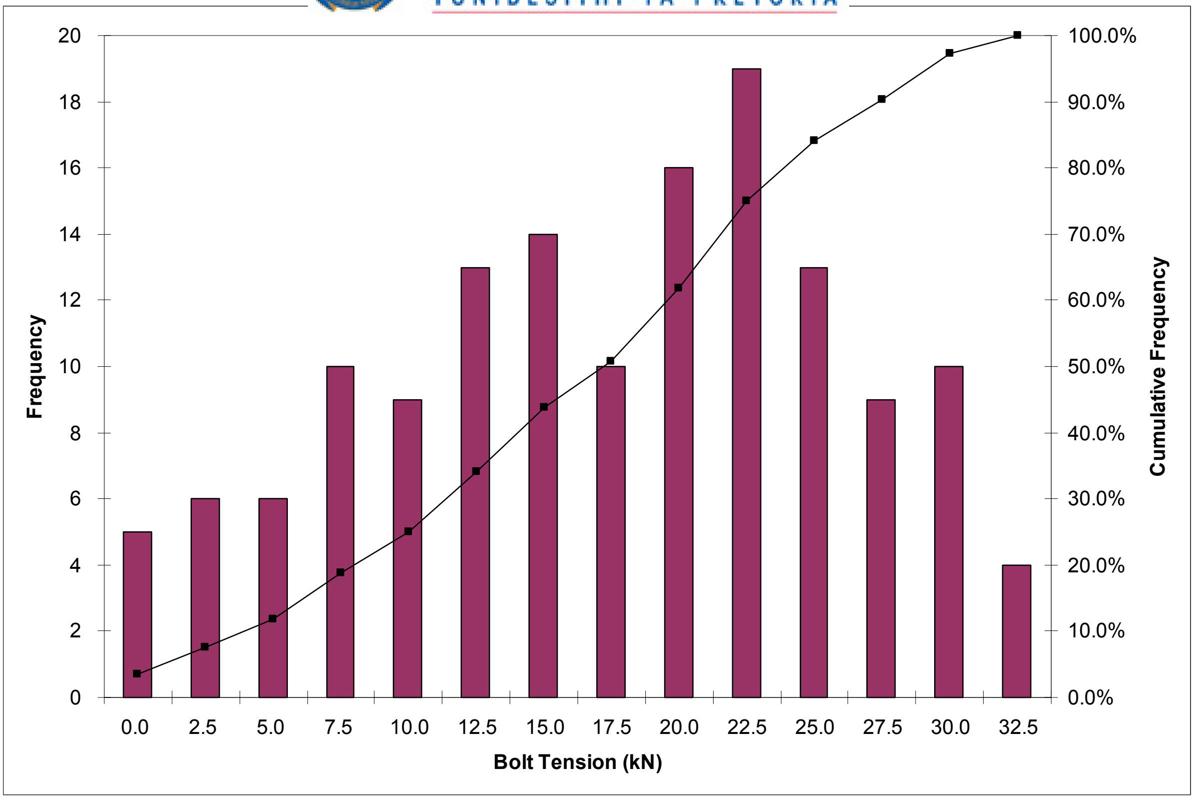


***Figure 7-17 Distribution of roof bolting tensioning results***

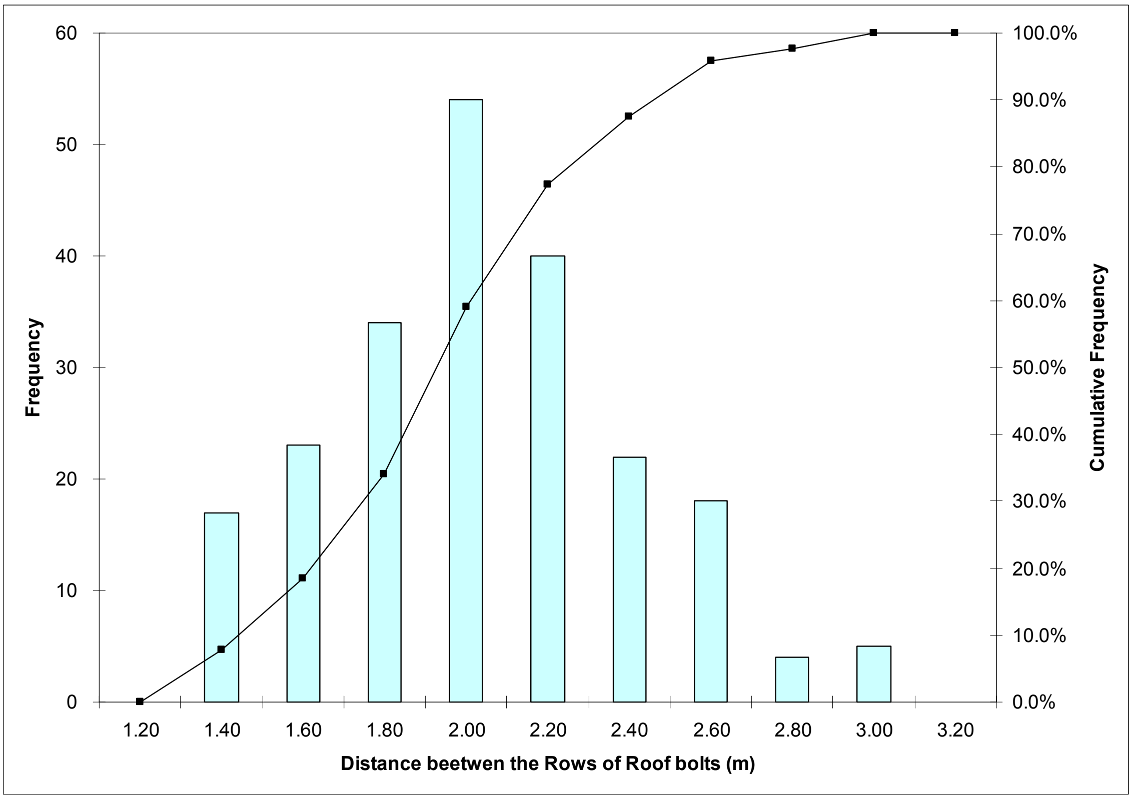


***Figure 7-18 Distance between the roof bolts measured in the experiment site***

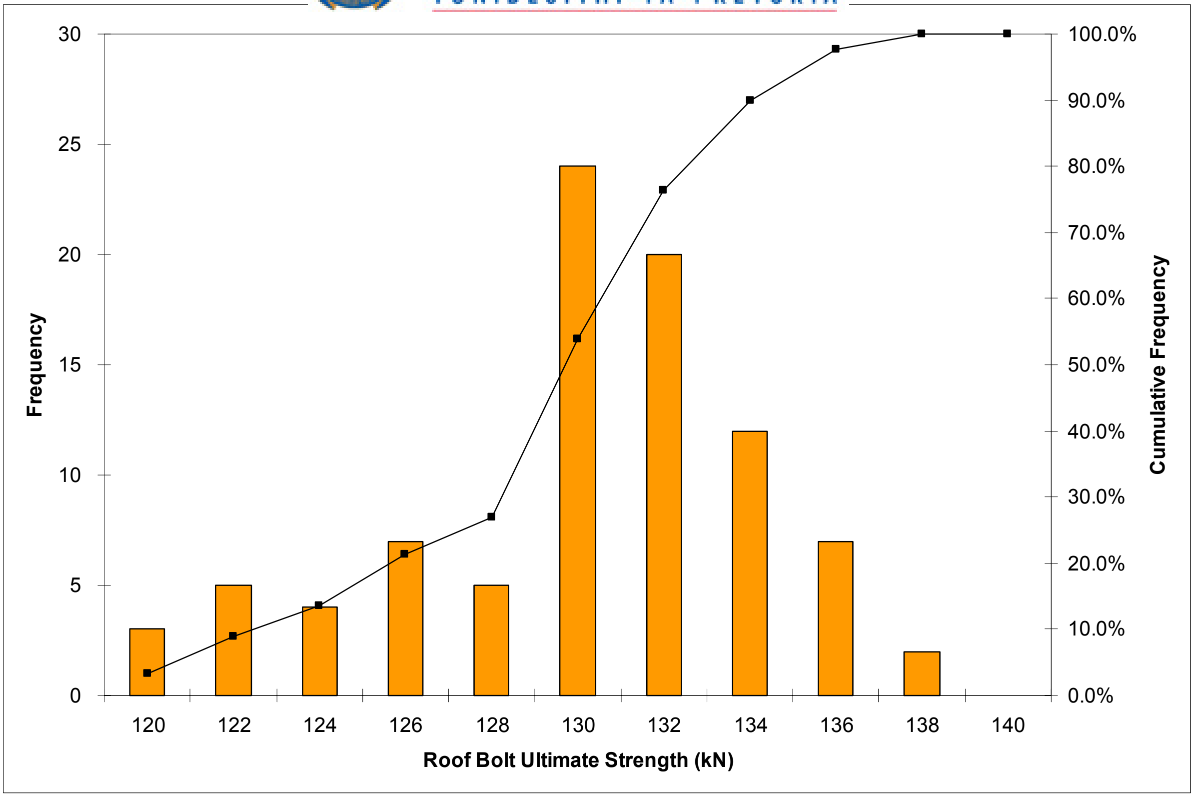


***Figure 7-19 Roof bolt ultimate strength***

An extensive laboratory testing programme was also initiated to determine the tensile strength of the immediate and upper coal layers with the aim of determining the applicability of suspension and beam building mechanisms as well as the stability of the immediate roof between the roof bolts. Additional information such as the unit weights of coal and shale were also determined from these laboratory tests. The distribution of tensile strength of coal as obtained from the Brazilian Tensile Strength tests is shown in Figure 7-20. Figure 7-21 shows the distribution of unit weights of the immediate and the upper coal layers determined from these laboratory tests.

Due to the lack of information at the mine regarding the coefficient of friction between the layers in the roof, the data presented in Table 7-5 was used in this study. Figure 7-22 illustrates the distribution of the data given in Table 7-5.

A summary of the information presented above is given in Table 7-8 together with the additional information obtained from the mine.

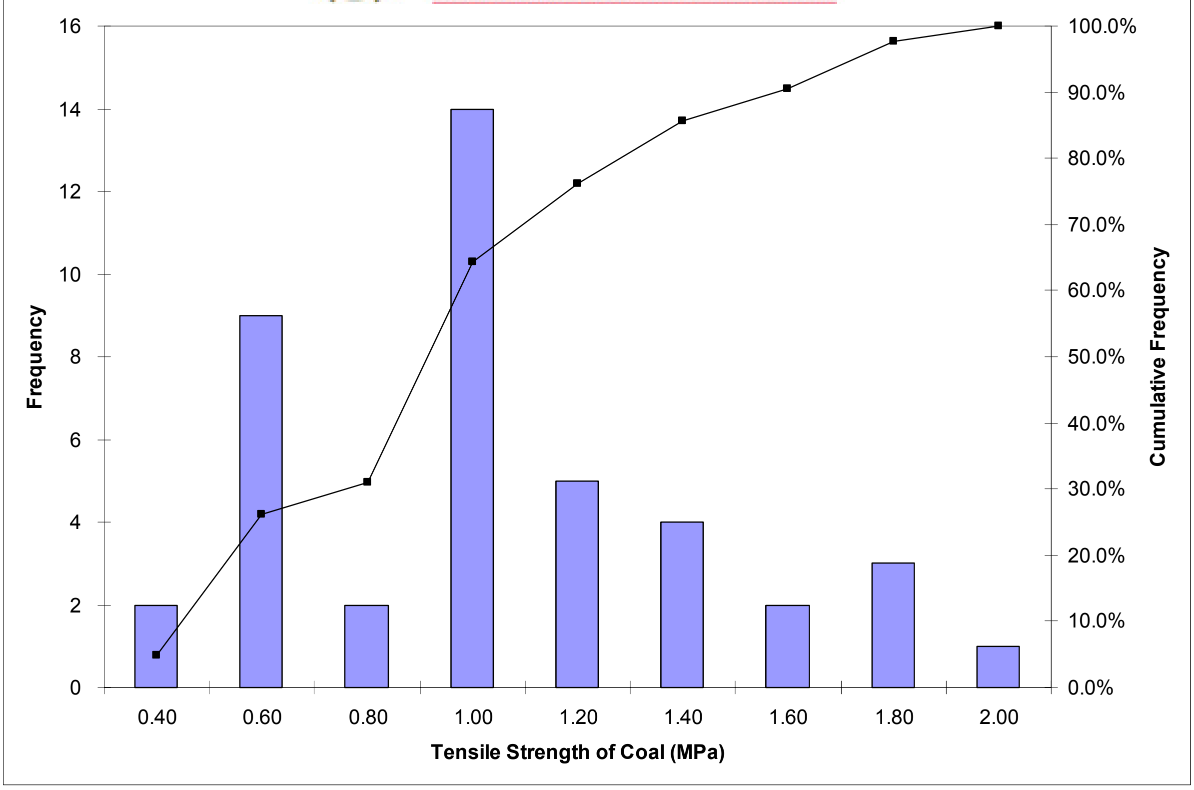


*Figure 7-20 Distribution of tensile strength of coal used in the analysis*

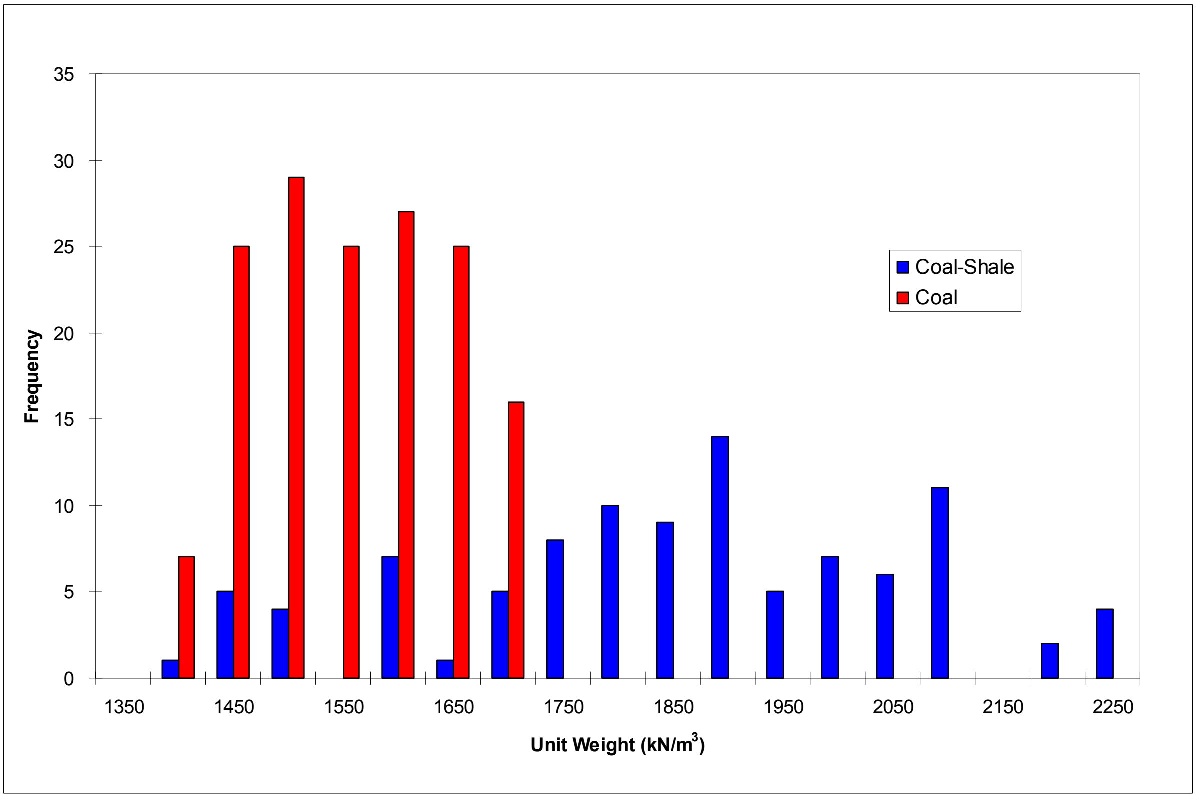


*Figure 7-21 Unit weights of the immediate and upper coal layers*

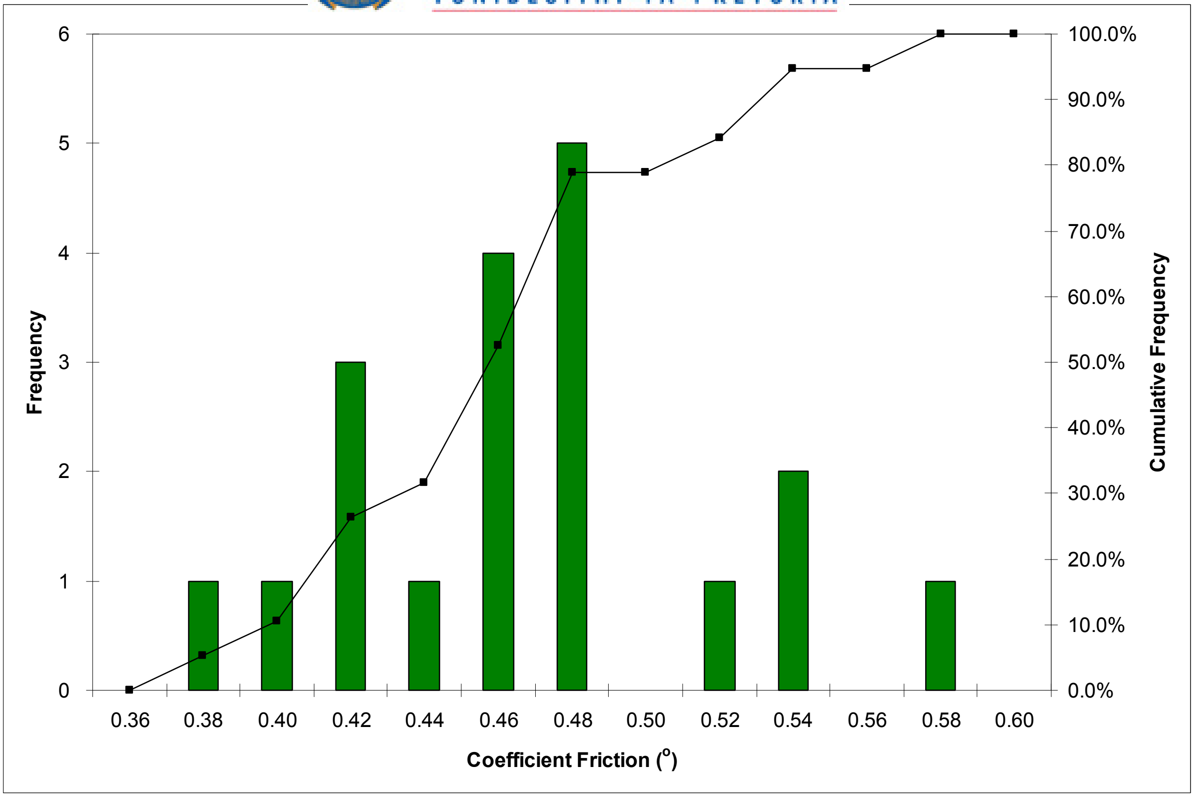


***Figure 7-22  Distribution of coefficient of friction between the layers***

***Table 7-8  Summary of information used in the analysis***

| | Number of samples | Minimum | Maximum | Average | Mode |
|---|---|---|---|---|---|
| Height of roof softening (m) | 93 | 0.2 | 1.6 | 0.7 | 0.6 |
| Bord widths (m) | 129 | 5.3 | 7.5 | 6.5 | 6.5 |
| Thickness of immediate layer (m) | 43 | 0.1 | 1.6 | 0.8 | 1 |
| Thickness of upper coal layer (m) | 43 | 1.5 | 3.3 | 2.5 | 2.1 |
| Bond strength (kN/mm) | 46 | 0.2 | 0.6 | 0.4 | 0.4 |
| Bolt tensioning (kN) | 145 | 0 | 32 | 16.4 | 20 |
| Distance between the rows of roof bolts (m) | 217 | 1.3 | 3 | 2 | 2 |
| Roof bolt ultimate strength (m) | 209 | 119.3 | 137.8 | 129.3 | 126 |
| Unit weight of immediate layer ($MN/m^3$) | 99 | 1382.8 | 2214.4 | 1835.3 | 1900 |
| Unit weight of upper coal layer ($MN/m^3$) | 154 | 1380.9 | 1669.7 | 1530.1 | 1531.2 |
| Coal tensile strength (MPa) | 40 | 0.4 | 1.8 | 1 | 1.2 |
| Coefficient of friction between the layers | 19 | 0.4 | 0.6 | 0.5 | 0.4 |
| Coalfield | Witbank | | | | |
| Seam | No 2 | | | | |
| Mining height | 3.0 m | | | | |
| Mining method | Continuous miner bord and pillar, 9 road section | | | | |
| Depth | 47 m | | | | |
| Pillar widths | 9.0 m | | | | |
| Number of bolts in a row | 3 | | | | |
| Cut out distance | 8.0 m | | | | |

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Regarding the input parameters presented above the following comments can be made:

- The input parameters can be divided into two distinct groups, namely uncontrollable parameters (representing the ground conditions, i.e. height of roof softening, thicknesses of the immediate and the upper roof layers, unit weight of rock, rock tensile strength and coefficient of friction between the layers) and controllable parameters (representing the mining practice, i.e. bord width, the distance between the roof bolts, bolt tensioning, strength of roof bolts).
- Uncontrollable parameters are the true reflection of ground conditions present and cannot be changed.
- Controllable parameters are however the true reflection of the responses to those conditions and can be changed/improved to increase the probability of stability of the roof bolting systems.

7.8.1.1 Results

In order to determine the support mechanism using the above input parameters, the applicability of the suspension mechanism, as applied by the mine, was investigated using the input parameters presented above. A total of 20,000 Monte Carlo simulations were run using Equation [7-10] and the results showed that although the average safety factor of upper coal layer is 1.79, the PoS of suspension mechanism is only 92.6 per cent with a Reliability Index of 0.53, which is not acceptable according to criteria set in Section 7.3.1 (i.e., the minimum required PoS should be 99 per cent to use the suspension mechanism with confidence). Figure 7-23 presents the distribution of safety factors for the stability of the upper coal layer using the probability distributions presented in Table 7-7.

Nevertheless, in order to demonstrate the probability of failure using the suspension mechanism with the input parameters presented above, a further study into the applicability of the suspension mechanism is conducted.

As expected, the results showed that the overall PoS of suspension mechanism (PoS of upper component layer x PoS of bolts x PoS of sliding of roof bolts) is only 52 per cent (see Figure 7-24 for the distribution of safety factors in suspension mechanism). In other words, 48 per cent of the roof supported using the suspension mechanism with 1.2 m roof bolts will result in failure.

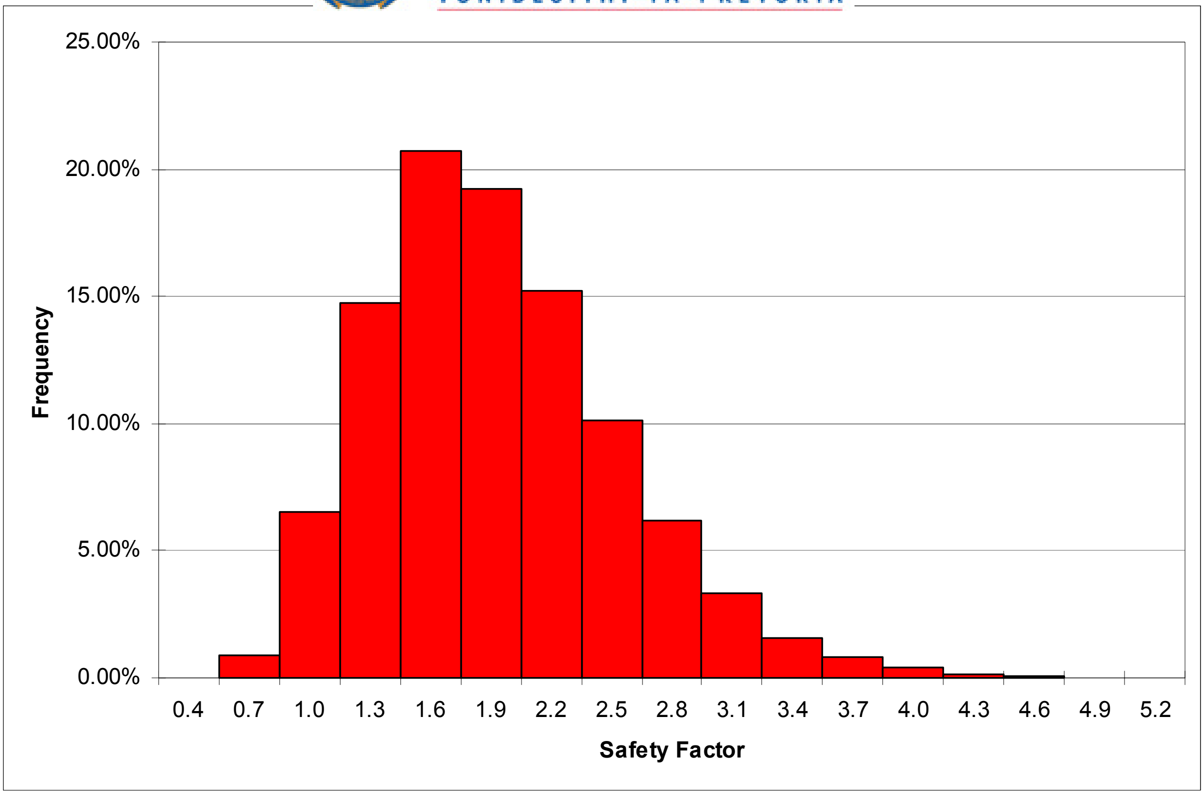


***Figure 7-23    Distribution of safety factors of upper coal layer in suspension mechanism***

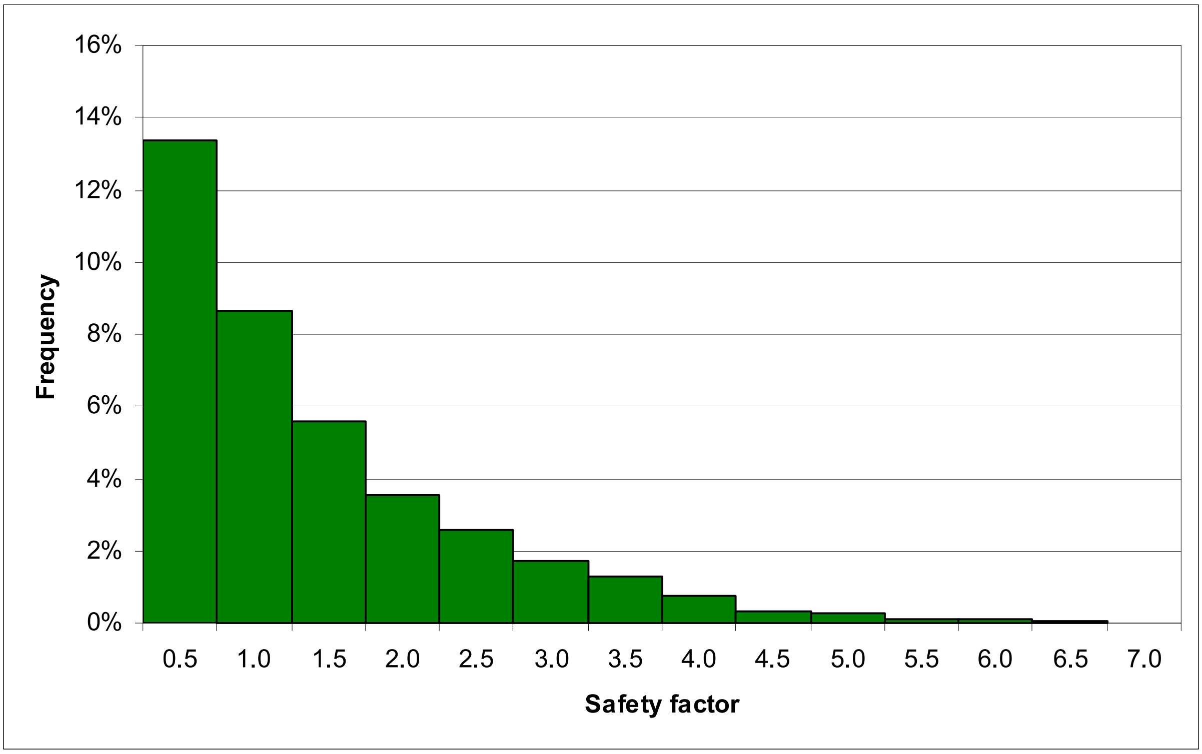


***Figure 7-24    Distribution of safety factors in suspension mechanism using 1.2 m long roof bolts***

Figure 7-25 shows the probability of stabilities and the reliability indexes for different lengths of roof bolts in suspension mechanism. As can be seen from this Figure that the maximum PoS that can be achieved is 92 per cent using 2.0 m long roof bolts, which does not meet the design criteria given in Table 7-4. Note that since the PoS of the suspension mechanism is dependent on the PoS of the upper coal layer, the maximum PoS that can be achieved for suspension mechanism is limited to 92.6 per cent.

From these analyses it is evident that the suspension mechanism, as it is currently used by the mine, is not the correct support mechanism for the roof conditions present at the mine. Therefore, beam building mechanism is recommended and a further study into the design of roof bolting system using the beam building mechanism is conducted.

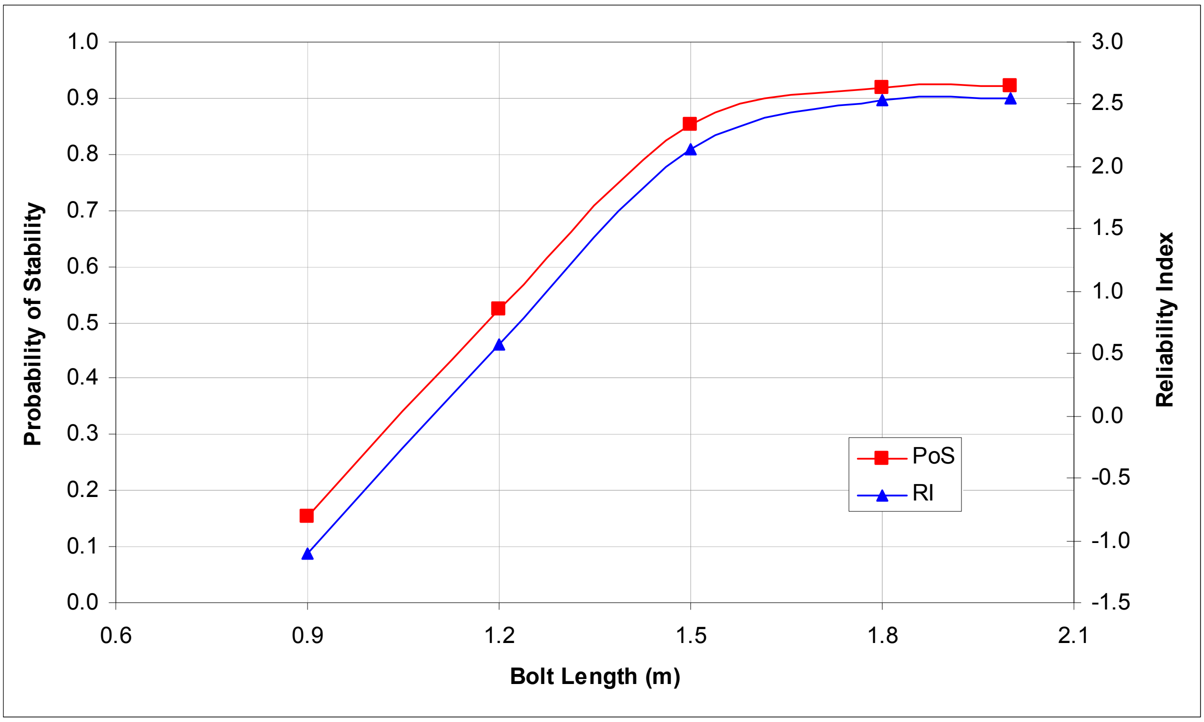


***Figure 7-25 PoS and Reliability Index for suspension mechanisms for different roof bolt lengths***

As a preliminary study, the mine's current support pattern, three bolts in a row with 2.0 m spacing was evaluated in beam building mechanism by assuming that the bolts are full-column. The probability of stabilities and the reliability indexes for different roof bolts lengths achieved from this study is presented in Figure 7-26. From this Figure it is evident that the current pattern used by the mine is not sufficient to achieve the required probability of stabilities even the bolts are full-column. Note that the overall probability of stabilities are presented in Figure 7-26 include probability of stability of shear loading, probability of bolt sliding and probability of bolt tension failures.

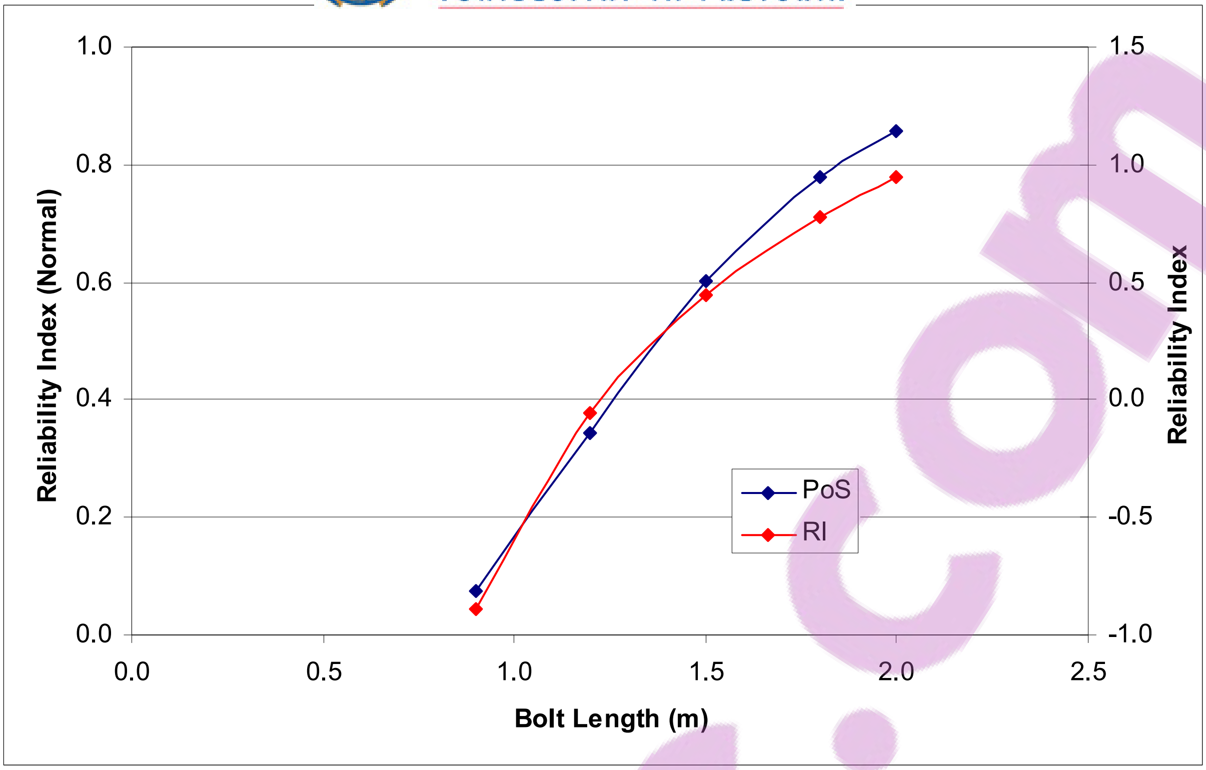


***Figure 7-26 Probability of stability and reliability index of different length roof bolts, 3 roof bolts in a row***

Table 7-9 shows the probability of stabilities and the reliability indexes achieved for 16 mm 4 and 5 roof bolt patterns using 2.0 m and 1.5 m row spacing. From this Table, the following *minimum* support patterns are recommended for different risk category areas:

- In moderately risk category areas:
  - four 1.8 m long roof bolts, 2.0 m row spacing
  - five 1.5 m long roof bolts, 2.0 m row spacing

- In serious risk category areas:
  - five 1.8 m long roof bolts, 2.0 m row spacing
  - four 1.5 m long roof bolts, 1.5 m row spacing

- In very serious risk category areas:
  - five 1.5 m long roof bolts, 1.5 m row spacing

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***Table 7-9 Stability analyses of different support patterns***

| | | Bolt Length (m) | | | | |
|---|---|---|---|---|---|---|
| Support pattern | | 0.9 m | 1.2 m | 1.5 m | 1.8 m | 2.0 m |
| 4 bolts in a row 2.0 m spacing between the rows | Probability of stability | 0.113 | 0.642 | 0.907 | 0.981 | 0.994 |
| | Reliability Index | -0.834 | 0.461 | 1.238 | 1.756 | 2.015 |
| 5 bolts in a row 2.0 m spacing between the rows | Probability of stability | 0.348 | 0.895 | 0.989 | 0.999 | 1.000 |
| | Reliability Index | 0.203 | 1.233 | 1.852 | 2.264 | 2.470 |
| 4 bolts in a row 1.5 m spacing between the rows | Probability of stability | 0.435 | 0.959 | 0.999 | 1.000 | 1.000 |
| | Reliability Index | 0.498 | 1.784 | 2.556 | 3.070 | 3.328 |
| 5 bolts in a row 1.5 m spacing between the rows | Probability of stability | 0.644 | 0.990 | 1.000 | 1.000 | 1.000 |
| | Reliability Index | 1.549 | 2.594 | 3.222 | 3.640 | 3.849 |

An important consideration at this stage is to conduct a simple cost analysis for different roof bolt systems to determine the financial viability of each system.

Once the bolting system is chosen and implemented, it is suggested that the support system should continuously be monitored and appropriate quality control procedures should be implemented (see Chapter 6.0 for details of quality control procedures).

As presented in Chapter 5, proactive rating systems (section performance and risk ratings) are effective in identifying support/roof performances. Therefore, detailed section performance and risk rating systems are also recommended in identifying the changing conditions, which may impact the support and the roof performances.

## 7.9 Conclusions

The ultimate aim of this Chapter was to develop a roof support design methodology that takes into account all natural variations exist within the rock mass and the mining process. This was achieved by adapting a probabilistic design approach using the well established stochastic modelling technique, which is widely used in civil and other engineering disciplines.

In the literature, it has been highlighted that one of the disadvantage of the probabilistic approach is the assumptions regarding the distribution functions. Using the data obtained throughout this thesis, the probability distributions of various input parameters have been established using the Anderson-Darling goodness of fit tests.

It is shown in this Chapter that the traditional deterministic roof bolt design methodologies provide some insight into the underlying mechanisms, but they are not well-suited to making predictions to roof support decision-making, as they cannot quantitatively address the risks and uncertainties that are inherently present.

An analysis of the data presented in Chapters 3 and 4 highlighted that for a 40 per cent increase in the span, taken across the diagonal of an intersection, relative to the roadway span, the magnitude of the displacement in the roof increased by a factor of four. The results also showed no evidence of a substantial increase in the height of the bed separated, potentially unstable roof strata, as is the case in the high horizontal stress driven beam buckling mechanism experienced in overseas coal mines. Analysis of the underground monitoring data also revealed that there is a good correlation between the underground measurements and simple beam theory, which has been used in the design of roof support systems for many years in South Africa. Therefore, in the development of the probabilistic approach, the deterministic approaches used in South Africa have been evaluated and improvements have been made, especially in the beam building mechanism.

Underground measurement data also showed that the maximum height of roof-softening measured in 54 sites in South African collieries is 2.5 m, which correlates well with the fall of ground data collected over 30 years in South Africa. The average height of roof-softening measured in these sites was 1.07 m.

The design approach established in this Chapter was applied to a well-defined case study in a colliery in the Witbank Coalfield, where the variations of all parameters that impact the roof and support behaviours were evident. Suspension mechanism has been used in this mine, which resulted in numerous roof falls. It has been shown using the input parameters collected from this mine that the suspension mechanism is not suitable for the conditions present. Therefore, the beam building mechanism was recommended for different risk category areas using four or five roof bolts with different lengths and row spacings.

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# Chapter 8.0

# Conclusions and recommendations

## 8.1 Conclusions

The ultimate aim of this thesis was to obtain an understanding of the fundamental mechanisms of roof failures and the fundamentals of roof bolting in South African collieries to provide guidelines and a design methodology for their amelioration. This was achieved through underground instrumentation, monitoring, testing and using well established stochastic modelling technique.

The conclusions arrived in this thesis are:

**Literature review (Chapter 2.0):**
Since the introduction of mechanical bolts in the 1940s, the amount of research into the understanding of the behaviour of roof bolts has been significant. Today, almost all coal mine roofs are supported with roof bolts in South Africa.

In the early years, the design of roof bolt patterns was based on local experience and the judgement of mining personnel. The suspension mechanism was the most easily understood and most widely used roof bolting mechanism. However, significant advances have been made over the last 20 years, in particular, the development of resin anchors, tendon elements, and installation hardware. These advances have resulted in an increase in the use of full column resin bolts.

The design of roof bolt patterns has also been improved, and four main rock reinforcement techniques have been developed: simple skin control, beam building, suspension, and keying. The geology and the stress levels determine the appropriate mechanism for a particular application.

The importance of tensioning roof bolts remains a subject of controversy. Due to the fact that the roof deformations in South African collieries are relatively small, it is recommended that tensioned roof bolts are beneficial in that they allow less roof deformation to take place after the support has been installed. However, if the bolting system is stiff enough, tensioning may not be required.

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Although there have been many studies into the support of intersections, a better understanding of rock behaviour in intersections is required.

Numerical models are useful in understanding roof and roof bolt behaviour; however, extensive laboratory studies are required for determining the input parameters for site specific conditions. The Australian technique, subsequently adapted in the UK, has proven that numerical modelling can be used to back analyse underground scenarios. Once the model is calibrated, then the results obtained from the numerical models can be used for design. No attempt has been made to develop a generic numerical model to be used in the design of roof support systems.

The selection of roof bolt type for different geological environments is well documented. However, the changing conditions underground must also be determined and the design and the support system have to be modified accordingly. Widespread instrumentation and vigilant visual observations are important for ensuring safety and stability in coal mines.

While the effect of roof bolt diameter on support performance is well understood, there is still controversy over the length of the roof bolts. It has been shown by Molinda et al. (2000) that the probability of roof failures increases with decreasing bolt length. Since skin failures (< 0.5 m thick) are more common in South Africa than larger roof falls (Canbulat and Jack, 1998, van der Merwe and Madden, 2002), short roof bolts and/or areal coverage for skin control may make up part of an effective support system.

This review also highlighted that different support design methodologies have been developed based on rock mass classification techniques, numerical modelling, instrumentation and monitoring and physical modelling. However, no attempt has been made to develop a probabilistic design methodology, which takes into account the natural variations exist within the rock mass and the mining process.

In conclusion, despite the fact that roof bolting has been the most researched aspect of coal mining, FOG still remain the major cause of fatalities in South Africa. There are no commonly accepted design approaches available for underground coal mines. Roof bolts were found to behave differently under different loading conditions, emphasising the importance of understanding the interaction between the roof bolts and the rock mass. The most important key to the design of a roof support system is a better understanding of roof behaviour and variations that can be encountered during extraction.

**Monitoring of roof and support behaviour (Chapter 3.0):**

The sonic probe extensometer, which was found to be the most accurate and reliable instrument capable of monitoring roof behaviour up to 7.2 m into roof, was used throughout the underground monitoring programme.

A preliminary study into the height to which the openings migrate in the roof (height of roof softening), i.e. height to which instabilities could occur was conducted. In all monitoring sites all the displacements measured in the roof were confined to within 2.5 m of the roof skin. The height of instability in the intersections was compared to that in the roadways with the elevation differences being converted to percentages. These differences were relatively small, varying between -5.0 and 33 per cent with an overall average of 13 per cent.

In the vast majority of cases the stable elevation in the roof was fully developed a short distance behind the face. In the drill and blast sections, the stable elevation was reached after a single blast, where the face advance increased the unsupported span to 3.0 m on average.

In the continuous miner sections, it was difficult to accurately determine at what point the stable elevation had fully developed. The only two monitoring sites that indicated obvious increases in the height at which displacement occurred in the roof as further mining occurred, were both in the partial column resin supported roof.

An investigation into the time effects of a static face indicated that close to a static face (within 0.5 m), the roof does not deform significantly. If a face remains static, the roof within its zone of influence (approximately 5.0 m away) experiences some degree of creep with time. An area of roof outside the zone of influence of the face (11 m away) is not affected by the face irrespective of whether it is stationary or be advanced.

The monitoring results also showed that there was no evidence of a dramatic increase in the stable elevations as is the case in the high horizontal stress driven beam buckling mechanism experienced in overseas collieries. It is thus concluded that in the sites monitored relatively high horizontal stress played little, if any role in increasing the deformations measured.

A roadway widening experiment was carried out to establish the critical roof displacements. The maximum width attained was 12 m at which stage $\pm$ 5 mm displacement was measured. No roof falls had occurred. However, in the same panel falls had occurred at 5 m widths. Also, falls took place in some of the areas where evidence of high horizontal stress had been noted. This indicates the significant variations that occur in a single mining area.

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During the monitoring period no roof falls occurred at any of the 29 sites and road widening experiment site, even where 12 mm displacements were measured. As a result it was not possible to try and establish critical roof displacement values for any of the geological regions.

In conclusion, these results showed that the roof conditions in South African collieries can be classified as gravity loaded beams.

**Effect of cut-out distance on roof performance (Chapter 4.0)**

The literature survey yielded little in the form of directly applicable research. It appeared that little work on determining roof failure per se as a function of cut-out distances has been done elsewhere. The limitations on cut-out distances were mainly due to other issues, like preventing underground workers being under unsupported roof and methane and dust control. Recent work done by researchers in the USA seems to indicate that extending the cut-out distance in the USA had little effect on roof stability, mainly because operators tended to reduce the cut-out distance under adverse roof conditions and only extend it if roof conditions were good.

During underground tests it was not possible to advance unsupported faces until failure occurred without exposing people to considerable risk. The next best was to monitor the universally accepted precursor to roof failure, which is roof deflection, under a range of different situations. This was done under the widest possible range within the constraints of time and funds, but it was still found that there were too many combinations of the variables that determine the roof deflection to derive complete answers.

The measurements were then complemented by numerical modelling, which affords the possibility to vary only certain parameters and keep the rest constant. It was then found that the underground observations fitted the patterns derived from the models and consequently there is a high level of confidence in the final conclusions.

The most important conclusion from this investigation was that once the face had advanced to a distance equal to twice the bord width, there was insignificant additional roof deflection with further face advance. This conclusion was confirmed by numerical modelling and is in line with the analytical beam solutions. For typical South African conditions, with bord widths in the range of 5.5 m to 7.2 m, the implication is that roof stability would not be adversely affected by advancing further than 11 m to 14 m. Majority of all of the total roof deflection that would take place, would occur during the first 11 m to 14 m of development. Therefore, if it is intended to limit roof deflection by restricting the cut-out distance, the cut-out distance would have to be limited to less than the bord width. During the investigation, it was observed that where adverse roof conditions existed, this was in fact done by underground personnel.

With regard to the effects of time on roof deflection, it could only be studied for the initial period of 48-hours following roof exposure. The reason for this was operational, as leaving faces for longer periods would have had an adverse effect on production and the sequence of mining. The instrumentation was usually done on Friday afternoons, preceding weekends during which faces would not be mined. It was found that the roof continued to deflect during that period, but that the amounts of deflection were not significant. However, it is still deemed necessary to support a roof as soon as possible, as even minute fractures resulting from the additional deflection may change the roof behaviour and eventually result in failure.

Results from one sonic probe monitoring hole showed that roof bolting had no remedial effect on roof deformations. Although the effect of roof bolting was specifically monitored by only one sonic probe monitoring extensometer, in general, the results showed that in none of the monitoring holes where roof displacements were recorded, was there any evidence of the roof being lifted due to installation of pre-tensioned roof bolts. This indicates that the roof bolt tensioning was not sufficient to close the pre-existing openings within the roof strata, where roof displacements were recorded. However, as indicated by the differences in the maximum displacements between the No 1 and No 2 holes, it may be concluded that roof bolting prevented further deterioration from taking place. In all the cases the displacements recorded by the No 1 holes (drilled next to the previously installed bolts) were less than those recorded by the No 2 holes (drilled in the centres of the unsupported areas) during the same monitoring period.

It was found that the lithological composition of the roof strata played a major role in the amounts of deflection that were recorded. Bedding separation was seen to occur at the positions where different strata types joined. This implies that the roof behaved like a set of composite beams of different characteristics. It was then also found that the amounts of deflection corresponded with the deflection that would be expected from gravity loaded beams.

Within the limits of horizontal stress that were present in the study areas (three of the sites exhibited obvious signs of elevated horizontal stress), the stress appeared not to have had a noticeable effect on roof deflection. This was confirmed by the numerical modelling. It was concluded that as long as the magnitude of the stress is insufficient to result in failure of the roof, it does not contribute meaningfully to deflection.

The implication of this is that the dilation in the roof is determined by bord width and roof lithology rather than cut-out distance, once the cut-out distance exceeds twice the bord width.

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This last conclusion is significant, as it offers the first possibility to predict roof deflection and consequently roof failure. The recommended process is as follows:

- Determine the thicknesses of the roof plates (or beams) by careful scrutiny of borehole logs.
- Calculate the maximum deflection for the desired road width using standard beam solutions.
- Calculate the induced beam stresses using the standard beam solutions.
- If failure is not predicted, the road width is confirmed.
- The cut-out distance should be determined by other considerations (ventilation requirements, etc), but at least it is known that there is little to be gained in terms of roof stability by restricting it to any distance that is greater than twice the bord width.

Roof deflection should then be monitored underground and the first warning sign should be where the amount of deflection exceeds the calculated amount, as that would indicate a change in conditions. Where that occurs, it would be prudent to reduce the cut-out distance, but even more so to reduce the road width.

Exemption from the 12 m restriction on cut-out distance may be obtained from the Principal Inspector provided that the mine can show that the risk to underground workers will not be adversely affected. This implies that a comprehensive risk assessment is required to obtain the exemption. The results of this investigation show that in general, the increased risk to roof instability due to extended cut-out distances is not a major factor and that the emphasis in the risk assessment should be on the other factors, namely the control of dust and methane and the probability of workers being under unsupported roof.

As with any matter relating to roof stability, it is recommended to base this type of exemption on a comprehensive hazard analysis. It is important to obtain a broad view, based on a general roof hazard plan that is required for other purposes as well.

The following steps are recommended for determining the effective cut-out distances for a given site:

1. General roof hazard plans should be drawn up for each section based on the borehole logs,
2. A detailed geotechnical analysis should be conducted. This analysis should include mapping of geological discontinuities, stress regime and roof lithology,

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3. The characteristic behaviour of the roof should be determined for the range of conditions, such as change in the thickness of the immediate roof layer, stress regime and bord widths,
4. Once the bord width and support method are established from the above, the cut-out distance can be determined as well. The most important control parameter is the bord width. If the bord width is chosen such as to result in deflection that is less than that resulting in failure using beam theory, there is little to be gained by restricting the cut-out distance.
5. With the previous steps in place, it remains to also stipulate a procedure that will prevent any person being under unsupported roof.
6. The support system that will be used in the section should also be monitored by continuing the monitoring after the installation of support. The critical factors in determining the support performance are the height of the instability into the roof, which determines the length of support, and the separations within the bolt horizon, which determine the stiffness of the support.
7. Once the cut-out distance is determined with regard to ground control, it should be checked against the ventilation and risk assessment plans.

The study area included one site where there was a high incidence of jointing, but in that area the effects of the jointing did not materialise in the measurements, most probably due to "experimental gremlins." The irony is that the roadways next to the one where the instrumentation was done suffered severe damage and the cut-out distances in those were reduced substantially by the operational crews. However, in the instrumented roadway, no damage occurred and the roof deflection was minimal.

Finally, logic dictates that the longer the cut-out distance, the higher the probability of encountering unexpected jointing with its accompanying negative effects on roof stability. This may be countered by instituting measures that will prevent personnel being under unsupported roof.

**Geotechnical classification techniques (Chapter 5.0)**

The purpose of this task was to evaluate and compare existing roof rating system that are used in South Africa and others that have been developed in other countries, and proposing the way forward for the development of a system that could be used universally on South African collieries to determine the roof conditions and quantitatively required support. The results showed that although many collieries have hazard plans, these plans do not readily quantify the mechanistic behaviour of the roof strata, they are mostly descriptive and are subject to different opinions. Therefore, they cannot be used for roof support design purposes. Furthermore, there is no uniform methodology behind the development of these plans, which makes it difficult for another person to apply them.

The CMRR could overcome most problems associated with the application of rock mass classification systems to coal mining. Also, in principal, the borehole core CMRR is a very similar system to impact splitter. However, due to its origin from case histories from the United States, certain modifications need to be applied to the system for the different conditions in South African coal mines. In the context of the South African coal mining industry, the following summary can be drawn regarding future improvements in the system:

- Requires exposure into the roof (underground CMRR only)
- Only the bolted height is rated. In South Africa, 2.0 m into the roof is the height that is usually rated.
- Although sets of joints have been considered in CMRR, single joints can have an influence and should thus also be included.
- Joint orientation is not included (underground CMRR only).
- Stress adjustment is required in the rating system to account for the influence of high horizontal stress (underground CMRR only)
- Blasting adjustment is not considered (underground CMRR only)
- Does not consider the position of soft or hard layers into the roof (both underground and borehole core CMRR)
- Requires skilled personnel to carry out ratings (both underground and borehole core CMRR)

Rating systems will continue to play an important role in coal mining practice. These systems should relate to the expected mode of failure of the strata for design and planning purposes. Underground rating and performance systems need to be incorporated with the roof rating systems into the overall ground control management to ensure adherence to design and overall mine standards. However, these systems cannot quantitatively determine the required support system in a given condition.

Although most collieries studied had some form of hazard identification systems in place, these systems are mostly descriptive in nature and therefore tend to be subjective. Moreover, these rating systems are used mainly for planning purposes, and not to determine the changing conditions underground. The systems have worked in some cases where one person had extensive experience at one mine. However, due to movement of personnel, there has been a loss of knowledge, insufficient documentation and a lack of updates of the local systems.

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Impact splitting test has been found to be an appropriate system to eliminate human error in core rating. The advantage of impact splitting over the individual colliery's geology based rating systems is its ability to readily quantify the roof instead of a mere description that can change from one person to another. Geology based systems have been developed from experience by mine personnel that certain soft or hard layers in the roof were a major cause of instability. During this study, impact splitting has shown a very good correlation with the geology based rating systems. The system can therefore be used during planning for good prediction of conditions ahead of mining. Furthermore, the system requires minimal training time and therefore does not require skilled personnel.

In conclusion, impact splitting tests, section performance rating and physical risk ratings systems developed in South Africa can be described as the most effective and appropriate for South African conditions. Impact splitting can readily quantify the roof conditions during planning with minimum subjectivity. Section performance and physical risk rating can be used for identifying changing conditions while mining and determining the best response to the different conditions.

It must however be noted that as shown in the previous chapters of this thesis that the roof lithology, stress regime and roof characteristics can change within meters in a production section. Therefore, in order to predict these changing conditions many boreholes required for a section, which would be very expensive and time consuming. In addition, borehole core based systems like the impact splitting are dependent on the quality of the core. Layers that are very weak or have very low cohesion can easily break during the drilling process. Geophysical techniques may therefore be more accurate in such cases for identification of these layers.

**Evaluation of roof bolting systems (Chapter 6.0)**

As part of this task all support components that are currently being used in South African collieries were evaluated.

Although a considerable amount of time was spent on the effect of the roofbolters on the performance of support systems, few trends could be observed in the parameters influencing the support performance. The study showed that there are no standards in South Africa for the parameters investigated (speeds, torque, and thrust). Underground testing showed that the variations in the parameters are greater than was previously believed. No correlation between the hole profiles and the parameters investigated could be discerned.

Nevertheless, this indicates that in South Africa, the installation quality of bolts varies significantly. Irrespective of design, the bolts are installed in completely different manners.

Unfortunately, there is no data available on the relationship between roof collapses and the quality of bolt installation. It is therefore impossible to determine empirically which support installation performs the best. This highlights a need for the best equipment performance for the best support installation to be investigated in detail. Such a study would assist in reducing the falls of ground and, therefore, the rock-related casualties in South African collieries. However, experience gained during the underground experiments showed that such work can only be done in a more controlled environment, such as with the laboratory.

Investigation into the effect of wet-dry drilling showed that both the bond strength and system stiffness were relatively greater for wet drilling than for dry drilling. The reason for this was not determined but is probably related to the surface condition of the holes and its influence on the adherence of the resin to the rock.

Five important elements of a bolting system have been identified. The impacts of those elements were qualified through short encapsulated pull tests.

The performance of roof bolts that are currently supplied to South African mines was also investigated. A series of short encapsulated pull tests in shale indicated that, on average, bond strengths obtained from the roof bolts supplied by Manufacturer “C” (referred to in the report) were approximately 18 per cent and 28 per cent greater than those obtained from the roof bolts supplied by Manufacturers “A” and “B”, respectively.

To determine whether variations in the profile of bolts supplied by the different manufacturers could account for the differences in performance, the bolt-core diameters and rib diameters from different bolt manufacturers in South Africa were measured.

The parameters that determine the contact strength between bolt and resin are rib-height, spacing between the ribs, and the rib angle. An investigation was conducted into the dimensions of roof bolts that are used currently. The results showed insignificant differences between the parameters that determine the bolt profile of South African roof bolts. Owing to the physical similarity between the bolts studied, it was not possible to determine the influence of these parameters.

The effect of rib angle was investigated and the results of a literature search showed that, as the rib angle increases away from normal to the bolt axis, so the pull-out load of the bolt decreases. It is therefore suggested that, in order to achieve relatively high pull-out loads, low rib angles on the bolts are required. This was confirmed by laboratory tests on different bolts

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with different rib angles in Australia (O'Brien, 2003). However, it is noted that lowering the rib angle may result in poor resin mixing performances.

Using a conceptual model to determine the effect of bolt profiles, it is shown that maximum pull-out loads can be achieved between the resin and roof bolt when:

- The ribs are relatively high;
- The distance between the ribs is relatively low; and
- The ribs are relatively thick.

The performance of resins that are currently being used in South African collieries was also investigated by means of short encapsulated pull tests. The results indicated that in sandstone the resin types from the two different manufacturers performed similarly. However, the strength of slow (5/10-minute) resins from both manufacturers was much lower than that of fast resins. It is concluded that in the majority of pull tests, failure took place at the rock-resin interface, indicating that the rock failed before the resin shear strength had been reached. It is therefore suggested that the strength of resin currently being used in South Africa is adequate. However, the stiffness of the system of which resin is a part should be determined by short encapsulated pull tests.

Again, the conceptual model developed to determine the effect of resin in the support system concluded that the failure characteristics of a roof bolting system will be determined by the shear strength of bolt, resin, and rock.

- The failure will take place at the resin-rock interface when the shear strength of the rock is lower than the resin (rock will fail).
- The failure will take place at either the resin-rock or resin-bolt interface when the resin shear strength is the lowest in the system.
- When the resin shear strength is the lowest in the system, the failure will be determined by the roughness of the hole and the bolt profile.

The test results showed that the reinforcing system using bolts from all four manufacturers performed almost identically in sandstone, but performed in different ways in the other rock types. The bolts from Manufacturer "A" performed slightly better in coal and shale rock types than the bolts from other manufacturers.

In order to investigate the effect of bit types, a series of short encapsulated pull tests were conducted. The results showed that the 2-prong bit outperformed the spade bit in sandstone

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and shale rock types. However, the average hole annuli obtained from the 2-prong bit were always greater than the spade bit. It is thought that this is because 2-prong bits drilled a rougher hole profile. Both the stiffness and the maximum load obtained from the 2-prong bits were greater than for the spade bits. These findings suggest that 2-prong bits are more effective in collieries than spade bits are.

The effect of hole annulus was also investigated. The results show that an annulus between 2.5 mm 3.8 mm resulted in the most effective bond strengths. Another interesting point is that as the annulus drops below 2 mm, it appears to have a negative effect on the bond strengths.

The effect of wet and dry drilling was also investigated by means of short encapsulated pull tests. The results showed that bond strengths and overall support stiffnesses are greater with the use of the wet drilling in all three resin types.

Tensioned versus non-tensioned bolts is one of the most discussed topics in roof bolting. A number of papers have been published on this topic in Australia and the US. An additional 25 short encapsulated pull tests were conducted to determine the effect of tensioning on bond strength. The results showed that non-tensioned roof bolts achieved significantly higher bond strengths than the tensioned bolts in sandstone and shale roofs. Similarly, the overall support stiffness of non-tensioned roof bolts was significantly greater than that of the tensioned roof bolts. This finding may be significant and therefore the effect of tensioning and non-tensioning on overall support system performance should be investigated in a control environment.

The effect of rock type on support performance was also investigated by means of a series of short encapsulated pull tests. The results from these tests highlight the very distinct differences between bolt system performances in different rock types. Sandstone was shown in the tests to produce significantly better results than shale and coal. From these results it can be concluded that rock type is one of the primary factors influencing the support system performance.

An investigation into the quality control procedures of support systems was also conducted. Quality control procedures for compliance with the design, support elements and quality of installation are presented in the thesis. Recommendations for improving quality control measures and for developing testing procedures for bolt system components, installation quality and resin performance are provided.

Most importantly, similar to stress regime, geology and roof characteristics presented in the previous Chapters, there is a significant variation in the performance of support systems using different support components in different geotechnical environments. Therefore, it is concluded that a deterministic approach is not adequate for a roof bolting system design in such a complex system. A probabilistic approach is required in order to take all these variations into account.

**Roof support design methodology (Chapter 7.0)**

The ultimate aim of this Chapter was to develop a roof support design methodology that takes into account all natural variations exist within the rock mass and the mining process. This was achieved by adapting a probabilistic design approach using the well established stochastic modelling technique, which is widely used in civil and other engineering disciplines.

In the literature, it has been highlighted that one of the disadvantage of the probabilistic approach is the assumptions regarding the distribution functions. Using the data obtained throughout this thesis, the probability distributions of various input parameters have been established using the Anderson-Darling goodness of fit tests.

It is shown in this Chapter that the traditional deterministic roof bolt design methodologies provide some insight into the underlying mechanisms, but they are not well-suited to making predictions to roof support decision-making, as they cannot quantitatively address the risks and uncertainties that are inherently present.

An analysis of the data presented in Chapters 3 and 4 highlighted that for a 40 per cent increase in the span, taken across the diagonal of an intersection, relative to the roadway span, the magnitude of the displacement in the roof increased by a factor of four. The results also showed no evidence of a substantial increase in the height of the bed separated, potentially unstable roof strata, as is the case in the high horizontal stress driven beam buckling mechanism experienced in overseas coal mines. Analysis of the underground monitoring data also revealed that there is a good correlation between the underground measurements and simple beam theory, which has been used in the design of roof support systems for many years in South Africa. Therefore, in the development of the probabilistic approach, the deterministic approaches used in South Africa have been evaluated and improvements have been made, especially in the beam building mechanism.

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The design approach established in this Chapter was applied to a well-defined case study in a colliery in the Witbank Coalfield, where the variations of all parameters that impact the roof and support behaviours were evident. Suspension mechanism has been used in this mine, which resulted in numerous roof falls. It has been shown using the input parameters collected from this mine that the suspension mechanism is not suitable for the conditions present. Therefore, the beam building mechanism was recommended for different risk category areas using four or five roof bolts with different lengths and row spacings.

## 8.2 Recommendations for future research

The following recommendations are made for future research as part of this study:

- Although there have been many studies into the support of intersections, a better understanding of rock behaviour in intersections is required.

- The effect of time is also important for stable and safer workings. Therefore, the effect of time on support performance needs to be evaluated.

- The different roof strata encountered in the coalfields are likely to have a significant influence on the deformation rates, and thus monitoring should be carried out in all the important geotechnical areas. The quantitative influence of slips, joints and other geological discontinuities is not well understood and should be evaluated.

- The Chapter on geotechnical classification techniques highlighted that borehole core based systems like the impact splitting are dependent on the quality of the core. Layers that are very weak or have very low cohesion can easily break during the drilling process. Geophysical techniques may therefore be more accurate in such cases for prediction of these layers and their accuracy and reliability need to be established. Innovations need to be made to reduce the costs of applying those techniques.

- It is recommended that further work on the effect of bolt profile on roof bolt performance be carried out, with the aim of achieving failure on the roof bolt-resin interface. It is also recommended that the quality of resin mixing should be investigated for different rib angles in order that the most effective rib angles for roof bolts can be determined. Unfortunately, because rib configurations in South African bolt types are very similar and because testing in this thesis took place in an underground environment (uncontrolled conditions), the effect of rib angle, rib height and thickness and spacing between the ribs could not be quantified. It is, therefore, suggested that these tests should be conducted in a controlled laboratory environment.

- The effect of tensioning, non-tensioning on support performance could not be established. It is suggested that a new testing procedure should be developed for testing the performance of tensioned bolts.

- The investigation into the support system design recommended that a study into the shear strength of full column resin bolt is required.

- One other important factor that affects the performance of a support system is the quality of support installation. This was not investigated as part of this thesis. New support installation techniques, such as the “spin-to-stall system”, helped collieries to improve the support installation practice. While the spin-to-stall system provides a simpler underground procedure, it is significantly more demanding on the roof bolting system components. The resin must provide sufficient time for adequate mixing and roof bolt insertion, then transform very rapidly from a liquid to a set state and develop high bond strength. The properties of the resin, the properties of the roof bolt, the breakout torque of the nut and other parameters are important in developing and optimising this new system. Development of an improved installation technique, which will minimise the human error in the installation of support and ensure all components of the bolting system are compatible, is therefore required to ensure the correct installation of support to improve the safety of the underground workforce.

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