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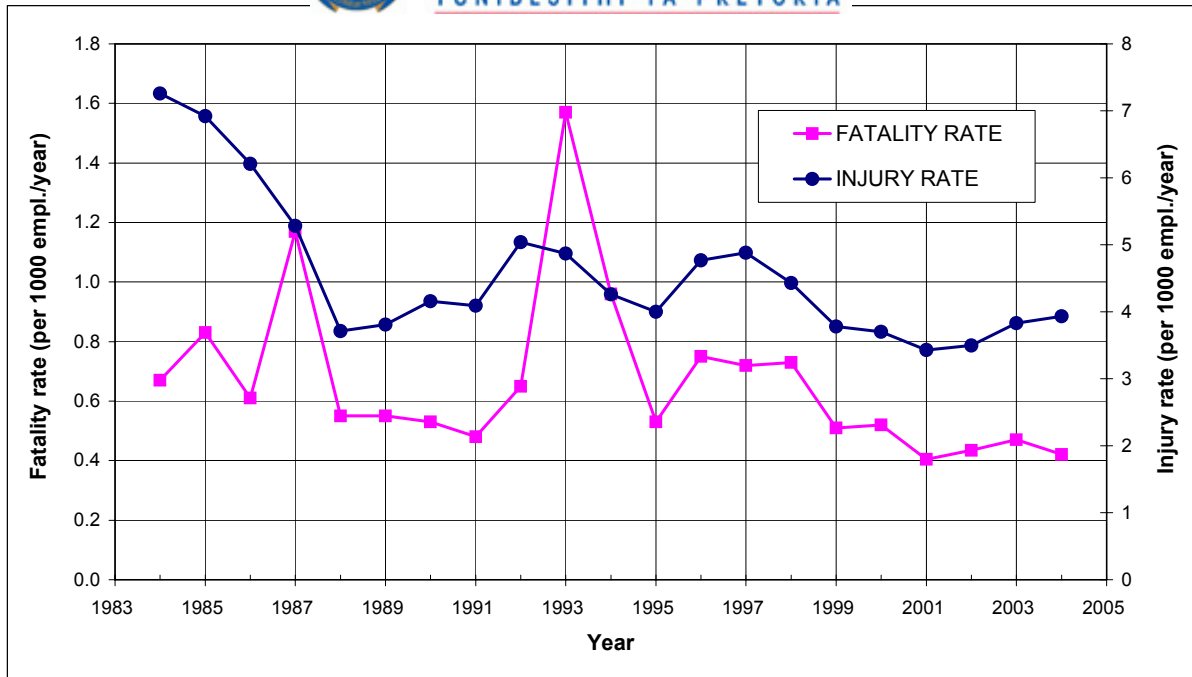
Figure 1-1   Fatality and injury rates in South African collieries for the period 1984 to 2001
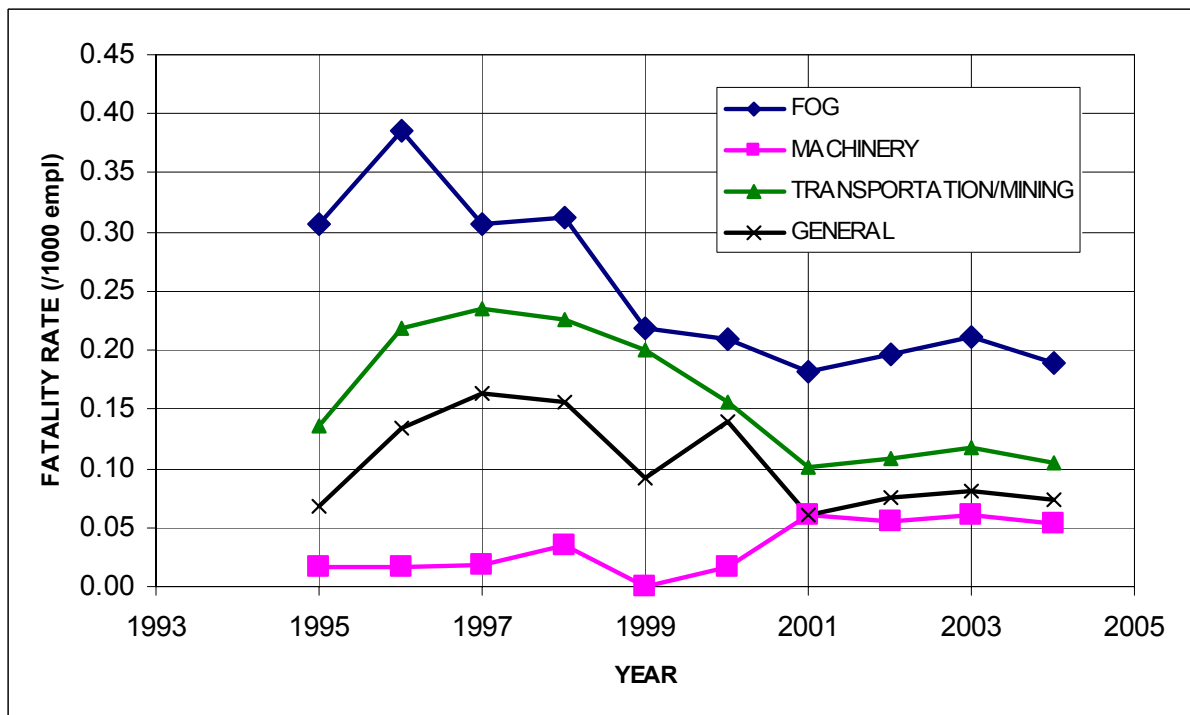

Figure 1-2   Cause for fatalities in South African collieries for the period 1995 to 2001

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.

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

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
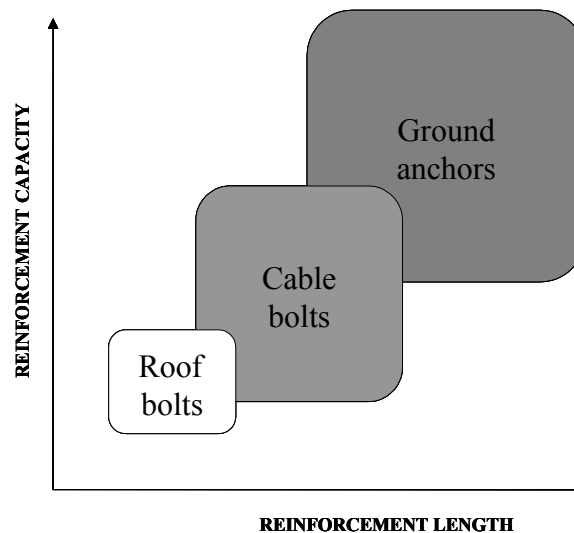Deep seated instability -   10-30 m-long elements or ground anchors

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.
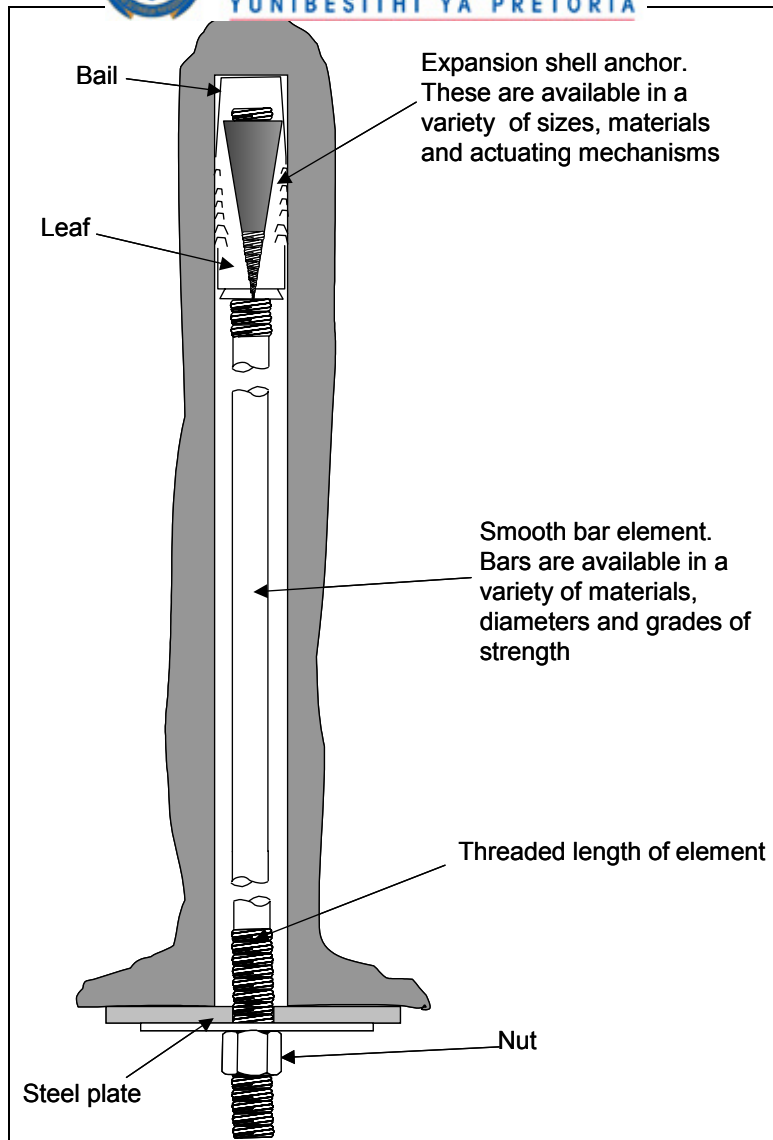
Bail

Expansion shell anchor.
These are available in a variety of sizes, materials and actuating mechanisms

Leaf

Smooth bar element.
Bars are available in a variety of materials, diameters and grades of strength

Threaded length of element

Nut

Steel plate

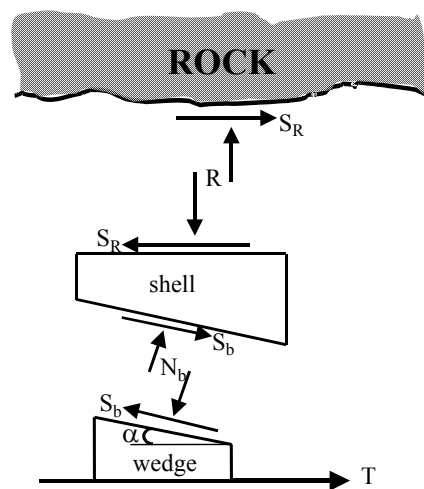*Figure 2-2   Mechanical anchor bolt*

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 (σ_r) and longitudinal (τ_r) stresses with the use of the following equations:

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

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

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

ϕ_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.

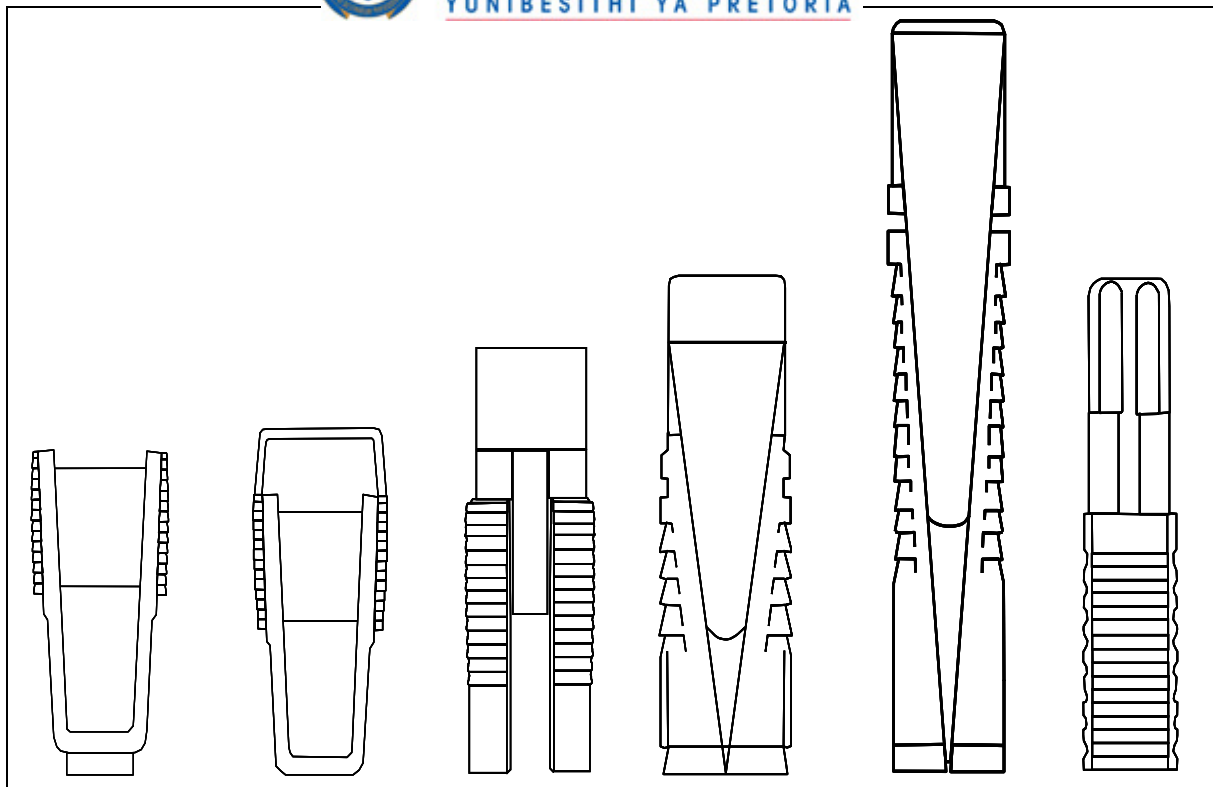Various types of expansions shells are shown in Figure 2-4.

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:
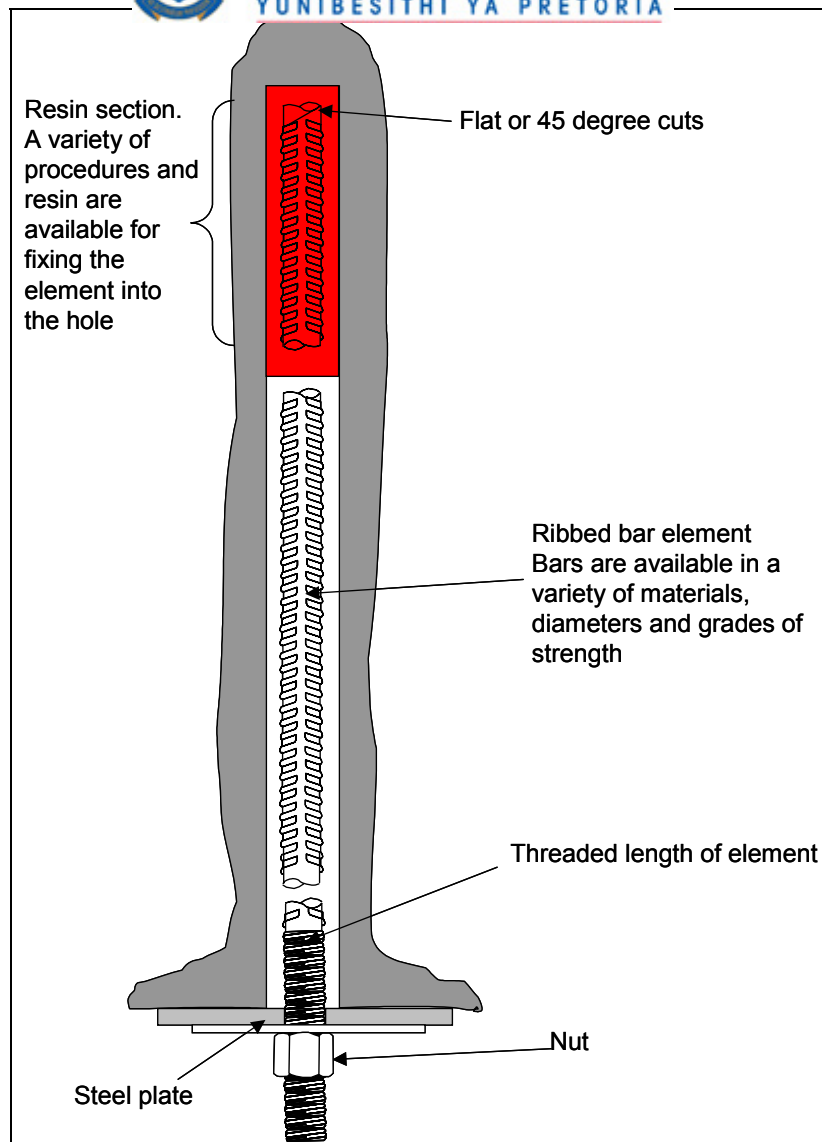
Figure 2-5   Point resin anchor

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.
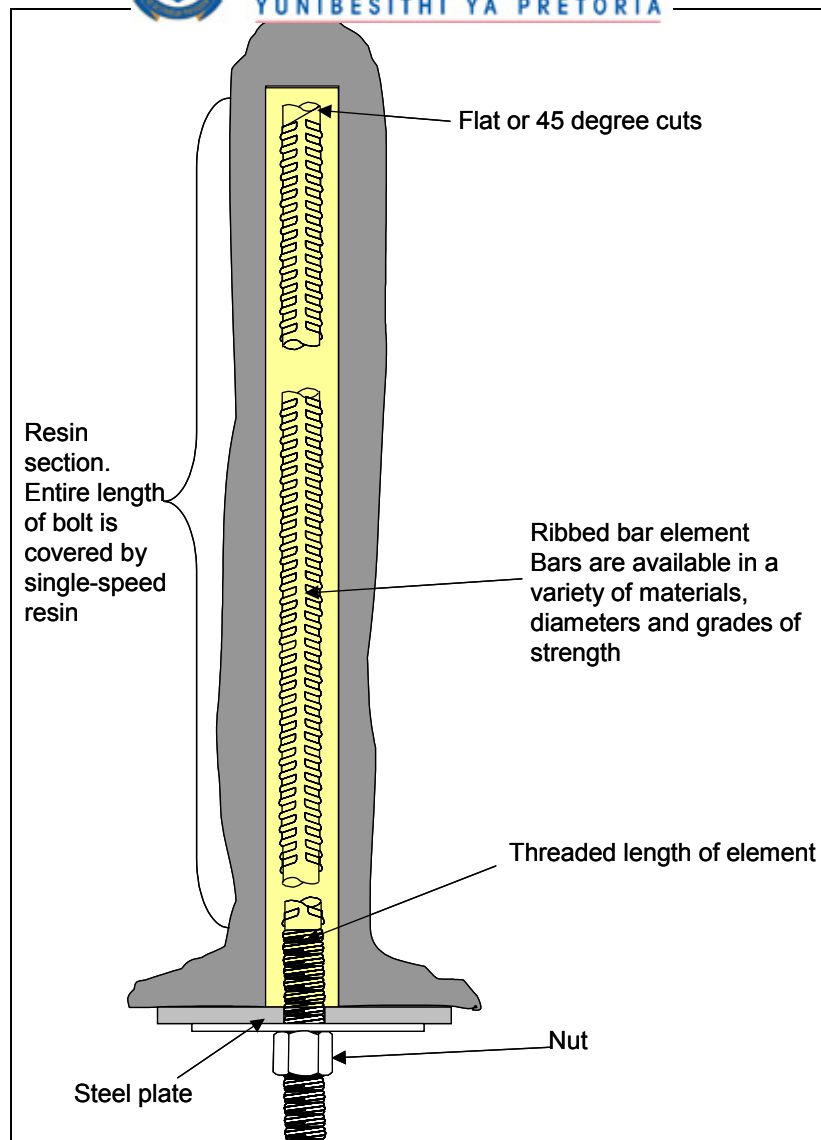
Figure 2-6   Full column resin bolt

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).
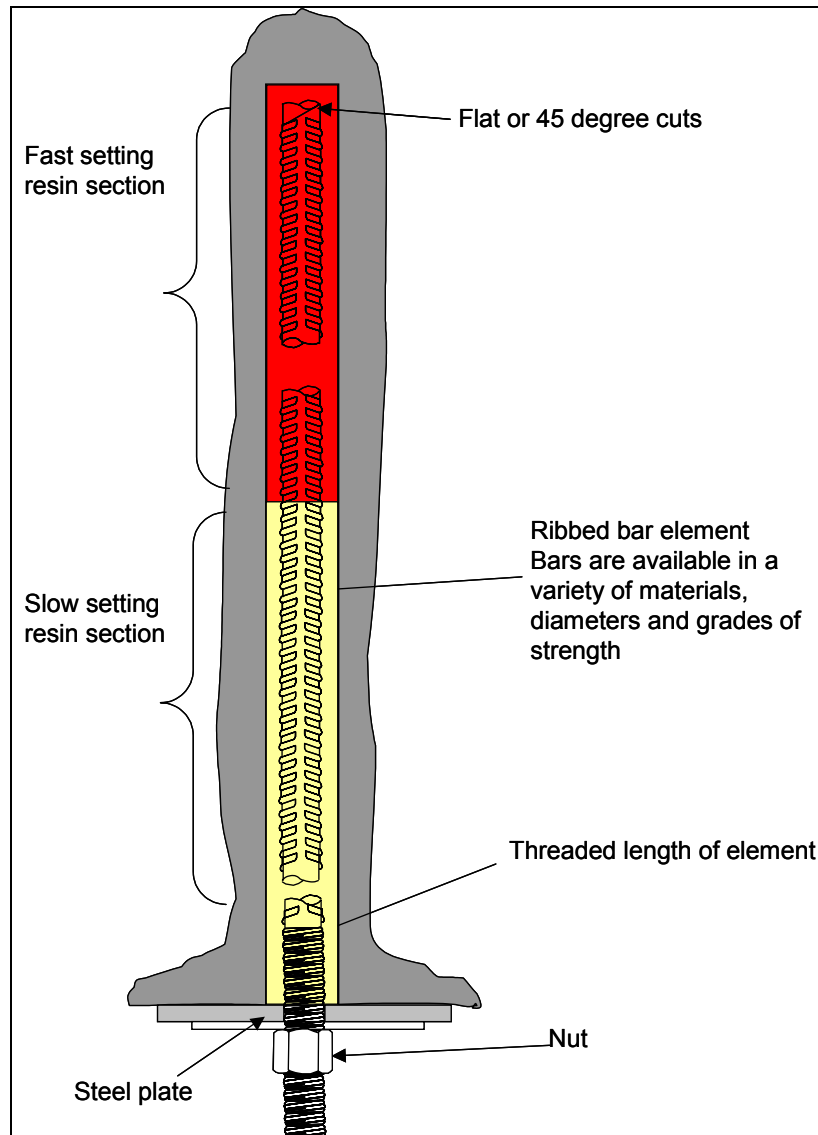
Fast setting
resin section

Flat or 45 degree cuts

Slow setting
resin section

Ribbed bar element
Bars are available in a
variety of materials,
diameters and grades of
strength

Threaded length of element

Nut

Steel plate

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

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).
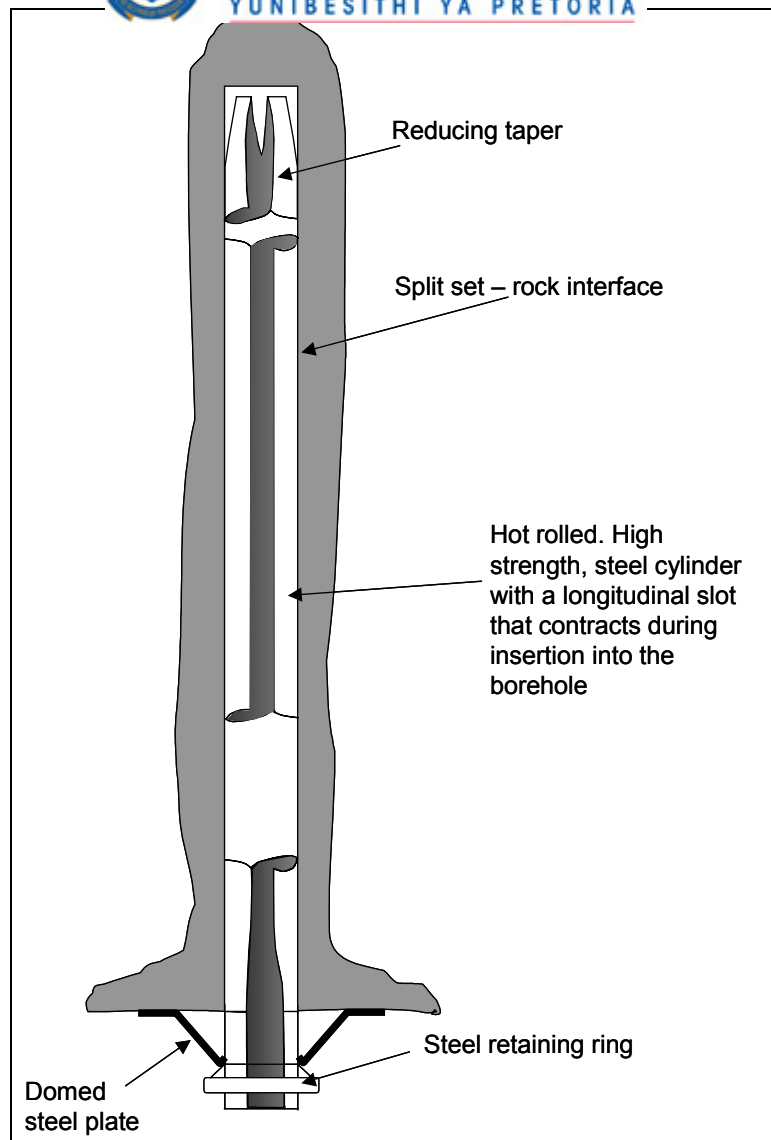
Reducing taper

Split set – rock interface

Hot rolled. High strength, steel cylinder with a longitudinal slot that contracts during insertion into the borehole

Steel retaining ring

Domed steel plate

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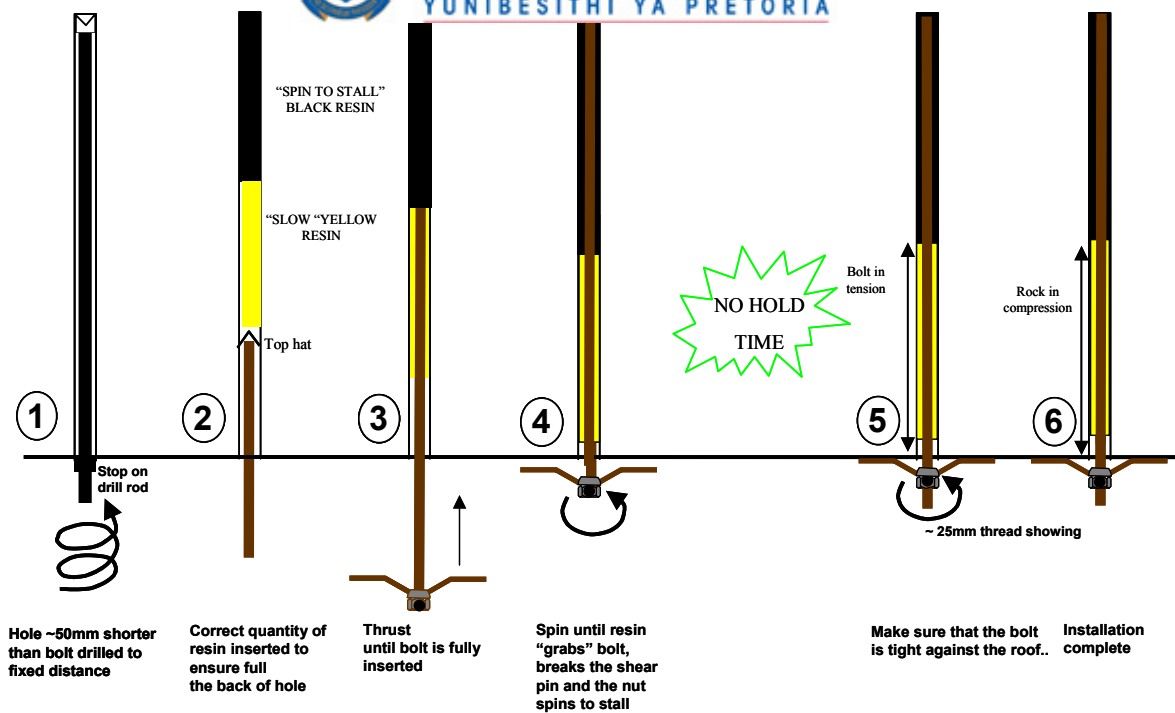
Figure 2-9   Spin-to-stall installation procedure (after Minney and Munsamy, 1998)

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.
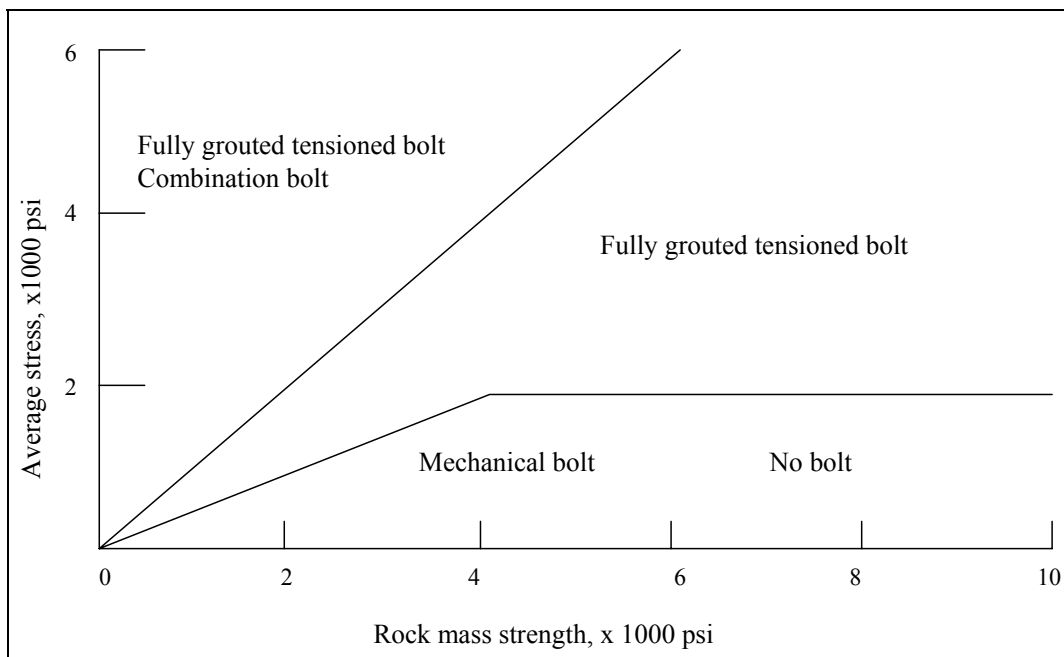

Figure 2-10   Selection of bolt type (after Maleki, 1992)

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

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 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 Split Set (short term)	Mechanical anchor
Thick layer of laminated material	Full-column resin bolts (slow/fast combination) Angled bolts	Resin point anchor Full-column resin bolts (single resin type)	Split Set Mechanical anchor
Thick layer of weak material	Full column resin bolts (slow/fast combination) Angled bolts Roof trusses	Full-column resin bolts (single resin type)	Resin point anchor Mechanical anchor Split Sets
High horizontal stress	Full-column resin W-straps Long anchors	Resin point anchor	Mechanical anchor
Burnt coal, ribsides	Split Set Wire mesh and Shotcrete	Dowels	Any resin anchor 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.

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).
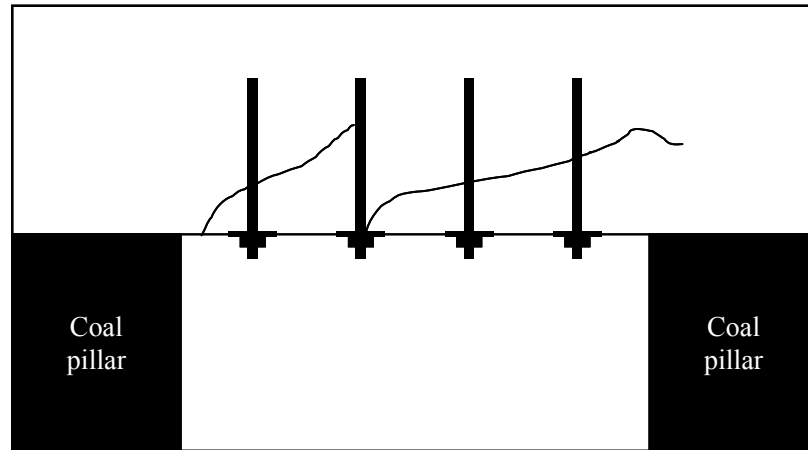
Figure 2-11   Simple skin support

2.3.2   Suspension mechanism

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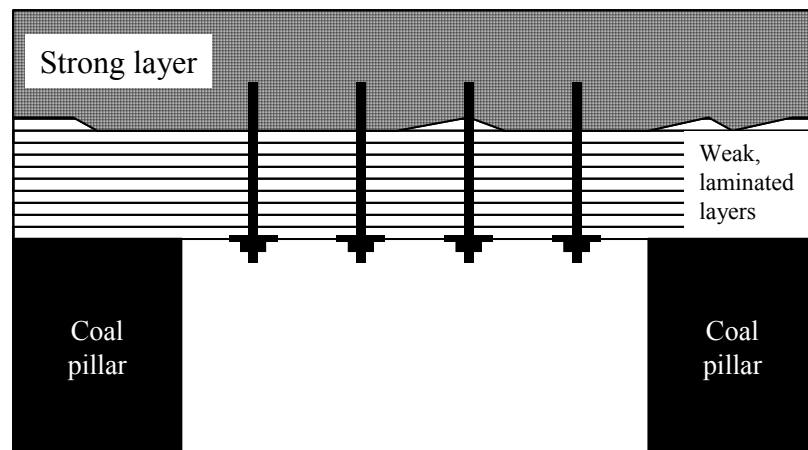

Figure 2-12   Suspension mechanism

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).
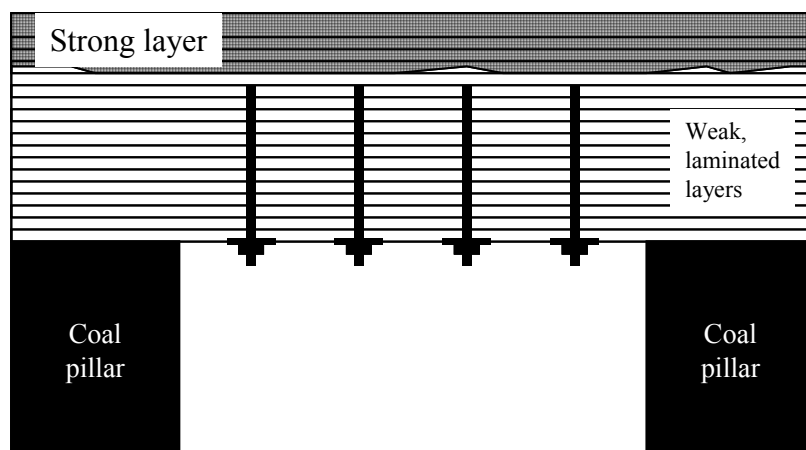

Figure 2-13   Beam-building mechanism

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).
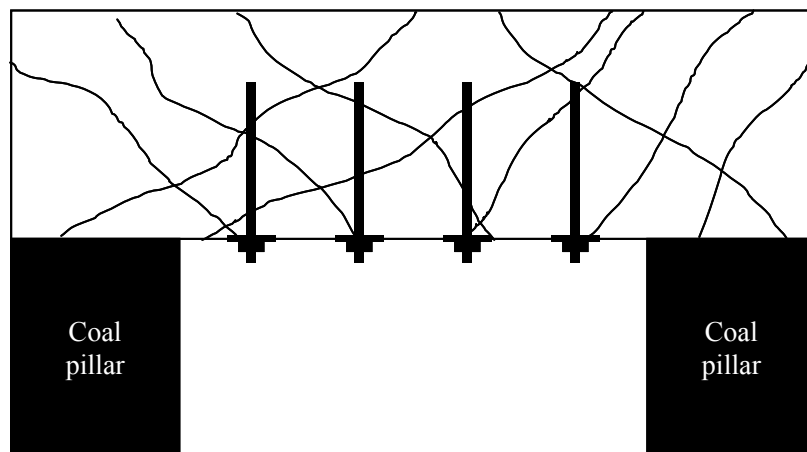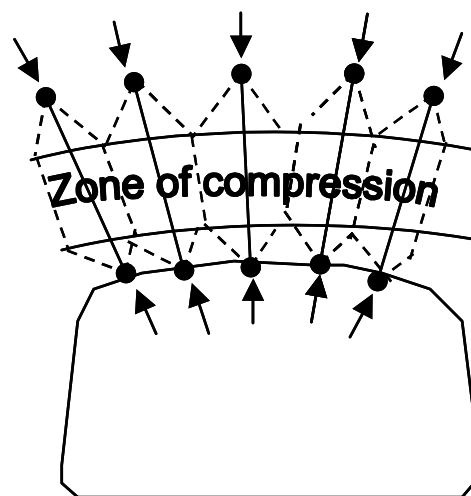

Figure 2-14   Keying effect of bolting

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.

$$\sum\nolimits_{i-1}^{n} AF_i > W$$ [2-5]

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

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

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

$$n = SF \frac{\rho g t}{P_f}$$ [2-7]

where, SF = Safety Factor

ρ = 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) $$\sigma_{xy} = \frac{\rho q L^2}{2t}$$ [2-8]

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

Maximum deflection (m) $$\sigma_{xy} = \frac{\rho q L^4}{32 E t^2}$$ [2-10]

where L = roof span (width of roadway) (m)

t = thickness of roof layer (m)

ρ = 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).

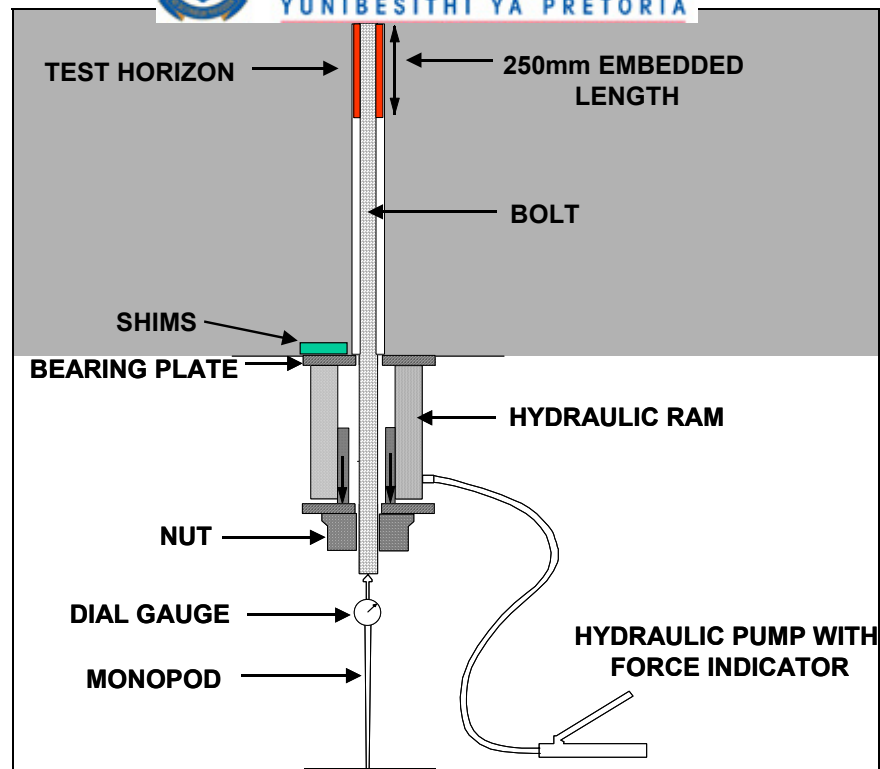Standard SEPT equipment is shown in Figure 2-16.

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.

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):

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

$$Contact\ Shear\ Strength,\ (\tau) = \frac{F}{\pi dl}\ [\text{MPa}] \qquad\qquad [2\text{-}12]$$

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

where F = Load to slippage (kN)

ΔF = Change in force (usually from 20 kN to 80 kN)

ΔD = Change in deformation (mm)

l = Anchorage length (250 mm)
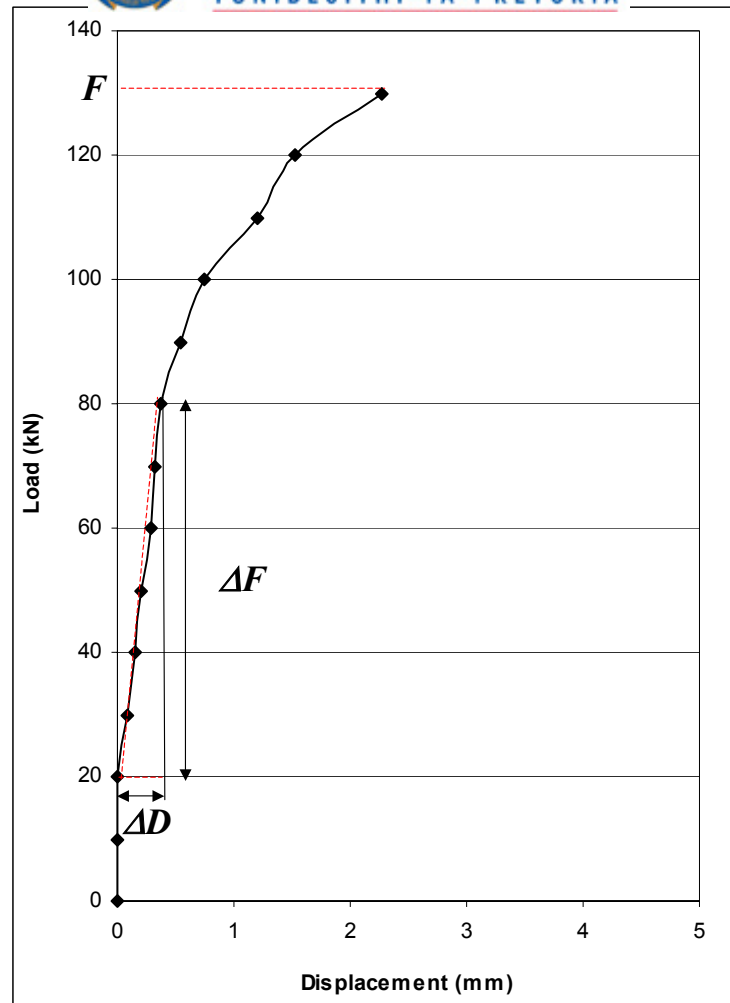
d = Hole diameter (mm)

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).
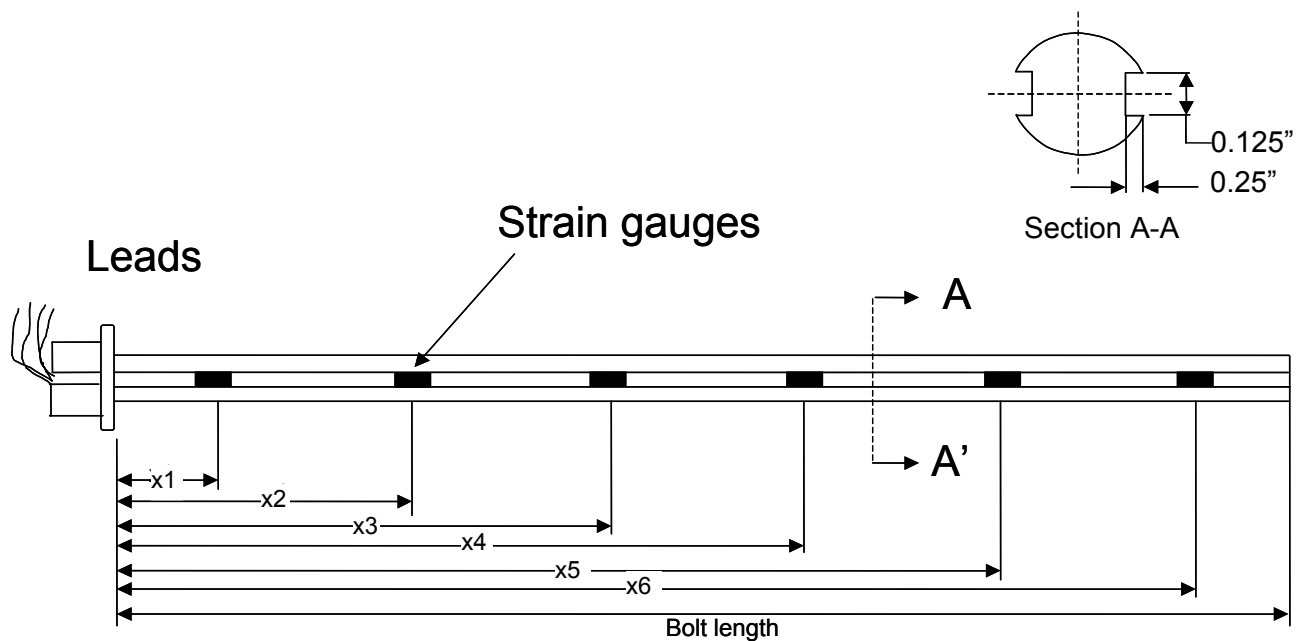

Figure 2-18   Instrumented roof bolt (after Signer and Jones 1990)

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.

35

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

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).
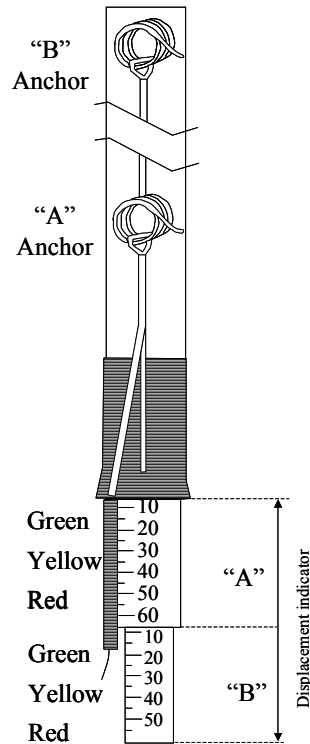

Figure 2-19   A tell-tales (after Altounyan et al., 1997)

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.
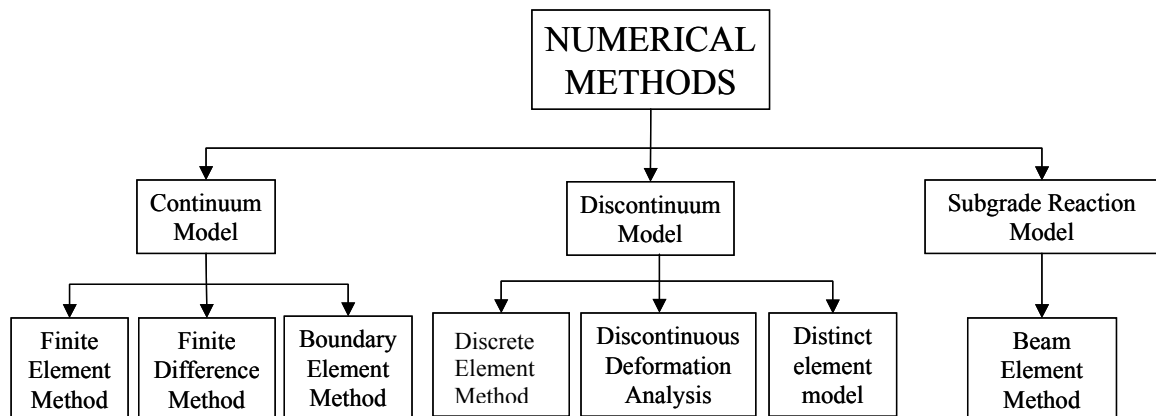

Figure 2-20   Numerical methods in rock engineering

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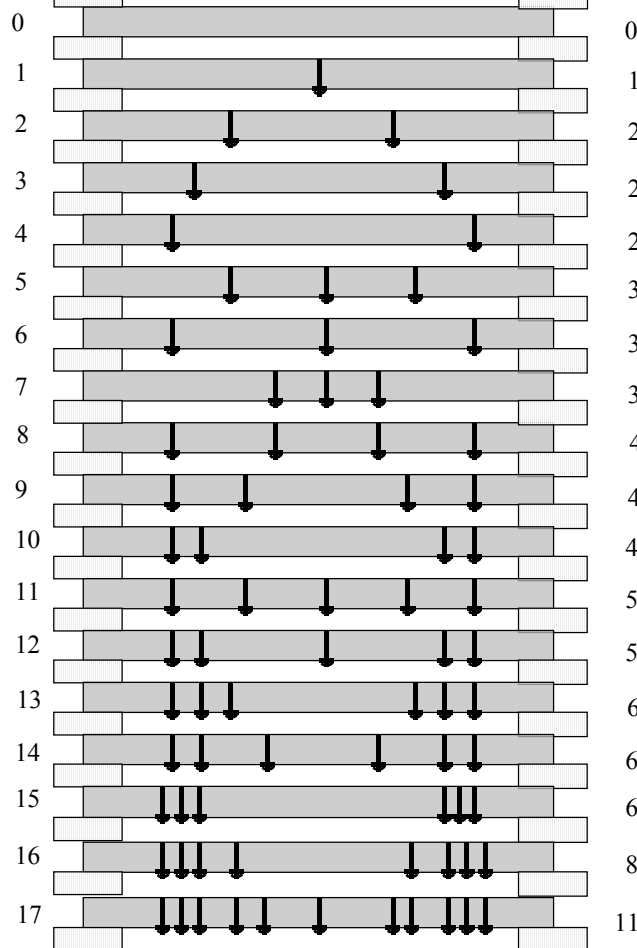
Figure 2-21   Bolt pattern (after Spann and Napier, 1983)

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

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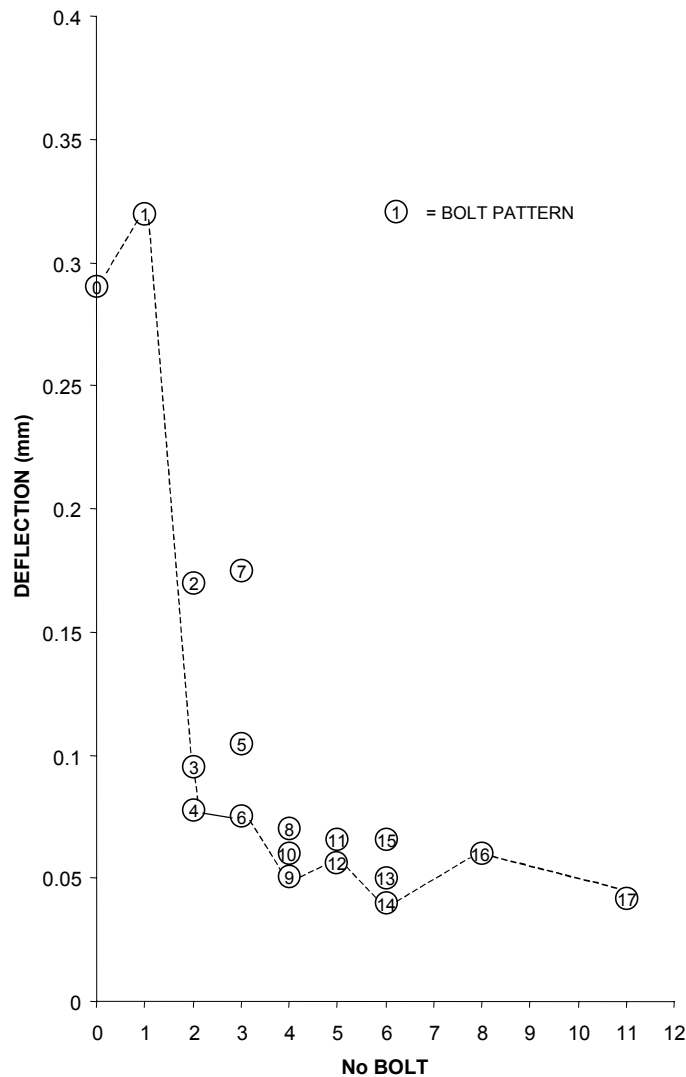

Figure 2-22   Deflection compared to number of bolts (after Spann and Napier, 1983)

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.

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, τ, 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:

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

where, δ = 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:

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

where, τ = mean shear strength (Pa)

P_f = yield load of bolt (N)

D = hole diameter (m)

L_c = capsule length (m)

Once τ 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.

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)

$$L_B = 1 \text{ m (strong homogeneous rock)}$$
$$L_B = (1/3 - 1/2)\, B \text{ (weak homogeneous rock)}$$
$$L_B \geq 1.5 \text{ m (strong stratified rock)}$$
$$L_B = (1/3 - 1/2)\, B \text{ (weak stratified rock)}$$

[2-16]

- Tincelin (1970)

$$L_B > 1/3\, B \text{ (Roadways)}$$
$$L_B \geq 1.25\,(1/3\ B) \text{ (strong stratified rock)}$$

[2-17]

- Lang and Bischoff (1982)

$$L_B = B^{2/3}$$

[2-18]

- Bieniawski (1987)

$$L_B = B/3$$

[2-19]

- Unal (1984)

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

[2-20]

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

- Mark (2001)

$$L_B = 0.12(I_s)\log_{10}(3.225H)\left\{\frac{100 - CMRR}{100}\right\}$$

[2-21]

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):

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

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:

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

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

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 σ 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:

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

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:

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

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

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

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.
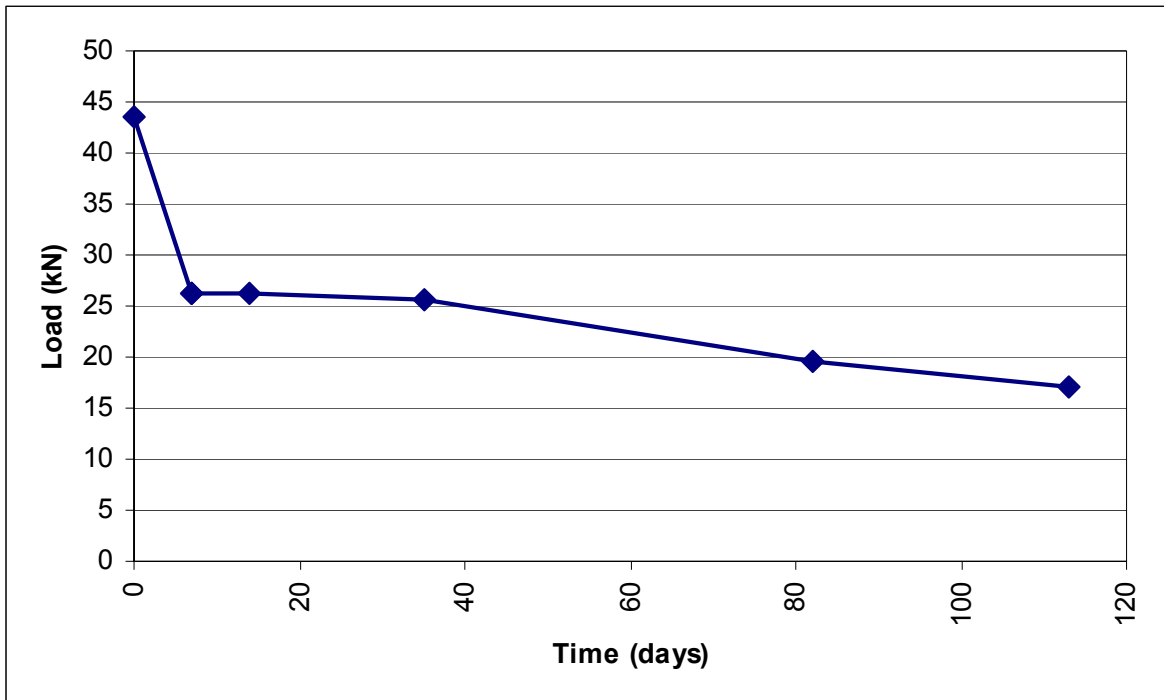

Figure 2-23   A typical plate load versus time in South African collieries (after Canbulat et al., 2003)

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):

$$K = \frac{A\,E}{L}$$

[2-28]

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-

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.

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.