Laboratory Investigation of Steel Cables and Composite Material Tendons for Ground Support

by

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Abstract

Composite material tendons are considered in this thesis as interior ground support for underground excavations. The physical and mechanical properties of composite tendons critical to mining requirements were identified. Preliminary laboratory investigations were also undertaken to evaluate their performance as fully grouted reinforcing tendons. Arapree, a flexible tendon, and Weldgrip, a solid tendon, are the two patented composite tendons found to be superior in pull out testing. Their load carrying capacity and ductility in the elastic range is similar to that of existing steel supports, although their post peak performance is different. Rigid composite bolts have more load sustaining capacity than their steel counterparts, while flexible composite tendons have less capacity than conventional seven wire flexible steel cable. The shear capacity of fully grouted Weldgrip bolts was found to be in the range of 100 kN. As part of studies of flexible tendons, conventional steel cable was also used to investigate other parameters, since it was found to be identical in pull out behaviour to composite flexible cable. It was found that the effect of grout strength and modulus, radial confinement and host medium strength and stiffness is significant in controlling the performance of steel cable. The laboratory investigation also found that different grouting materials, such as conventional to high strength cement grout and polyester resin grout greatly influence the pull out performance of both steel and composite tendons.

Résumé

Nous considérons dans cette thèse les tendons en matériaux composites comme support de base interne pour les ouvrages souterraines. Les propriétés physiques et mécaniques des tendons composites sont critiques lorsque les besoins des exploitations minières sont identifiés. Des études préliminaires en laboratoire ont été realisées afin d'évâluer leur performance en tant que tendons de renforcement. Arapree, un tendon flexible, et Weldgrip, un tendon solide, sont les deux seuls tendons composites brevetés ayant des capacités de résistance supérieures à l'arrachement. Leur résistance à la charge et à l'étirement dans le domaine élastique est similaire aux supports en acier existant, bien que leur performance de pointe soit différente. Les verrous rigides composites possèdent une plus grande résistance à la charge que leur équivalent en acier, alors que les tendons flexibles composites ont une capacité moindre que les câbles traditionnels à sept brins en acier. Il a été démontré que la capacité en résistance au cisaillement des verrous Weldgrip est d'environ 100 Kh. En tant que partie intégrante de l'étude sur les tendons flexibles, les câbles en acier conventionnels ont également été étudiés afin de déterminer d'autres paramètres, dans la mesure ou il a été de montre qu'ils avaient des capacités de résistance à l'étirement équivalentes aux câbles flexibles composites. Il a été établi que les effets de jointure et de modularité, le confinement radial, la résistance moyenne de base, et la résistance à la flexion sont significatifs pour le côntrole des performances des câbles en acier. Les tests en laboratoire ont également permis de montrer que différent matériaux de jointure, tels les ciments de jointure conventionnels à grande résistance et les résines de jointure en polyester ont une influence importante sur les performances des tendons en acier et en matériaux composites.

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Table of Contents

List of Figures

List of Tables

Chapter 1

Introduction

1.1 General

One of the main reasons for using support in underground mining is to maintain the inherent strength of the rock mass to support itself after it has been disturbed by an excavation. Support acts as reinforcing elements; it generally helps to transfer the weight of loose rock to the more intact and solid rock mass. The function of reinforcing elements is crucial to the stability of many underground and surface geotechnical structures during and after excavation. Ground instability may lead to increased cost in both repair work and delays. A proper design of a support system for the life span of a mine should consider the overall stability of the mine structure during ore extraction. It should also prevent any localized sudden failure within a rock mass, which may occur for example due to excessive deformation or rockbursting.

Support systems can be classified into two broad categories: internal and external support. These can be either active or passive. A support becomes active when stresses are induced in it at the time of installation. Therefore, such supports reinforce the rock mass structure by exerting an "induced" stress on the ground immediately after installation. The common examples are pretensioned rockbolts or cables, hydraulic props, expandable segmented concrete linings and powered supports for longwall faces. Active supports are applied in situations where an excavation is believed to cause excessive deformation in the ground and hence potential instability. An example of this situation is the separation of rock wedges or blocks from the rock mass.

Passive supports do not reinforce the rock mass immediately after installation but their effect is seen as subsequent mining activities take place. Examples of passive supports are steel arches, timber sets, composite packs and untensioned bolts (rockbolts, cable bolts, reinforcing bars, etc.). Cable bolts, in particular, interact with the complex and uncertain behaviour of the rock mass. For example, consider if they are installed prior to the excavation of a cut-and-fill stope. The stope is then mined in shifts by the drill and blast method. The reinforcing cables undergo various types of loading, including dynamic loading due to blasting, gravity loading due to massive broken rock blocks, and concentrated loading on small segments of cable due to separation or shearing of discontinuities.

External supports are generally of the passive type. These are placed around the boundary of the excavation to help restrain the movement of the rock walls and avoid the failure of a rock mass. Steel arches, wooden cribs and fiber reinforced shotcrete are some types of external support. Extruded concrete lining is considered an effective support in the case of permanent tunnels. Some degree of rock displacement almost invariably occurs prior to the installation of such lining. Because of its high material and labour cost, concrete lining is often replaced by shotcrete. Backfill is another common type of external passive support used in hard rock mining.

Internal supports continue to undergo technological development; their use is quite popular both in mining and civil engineering applications. The basic mechanism of internal support is to bond rock blocks together to maintain the overall stability of the rock mass around an excavation. Internal supports which are pretensioned at the time of installation are considered to be of the active type. Swellex, Split Sets, grouted bars and mechanical anchors are some of the common examples of internal support. Swellex and Split Set bolts operate by frictional resistance between the bolt and the rock wall. Fully cemented or resin grouted bolts depend on the bonding resistance of the grout for the transfer of load from the rock to the bolt. Mechanical anchors utilize a gripping mechanism at the toe and are installed with an end-plate to ensure proper anchorage. A general classification of internal support is presented in detail by Scott (1987); as shown in Figure 1.1.

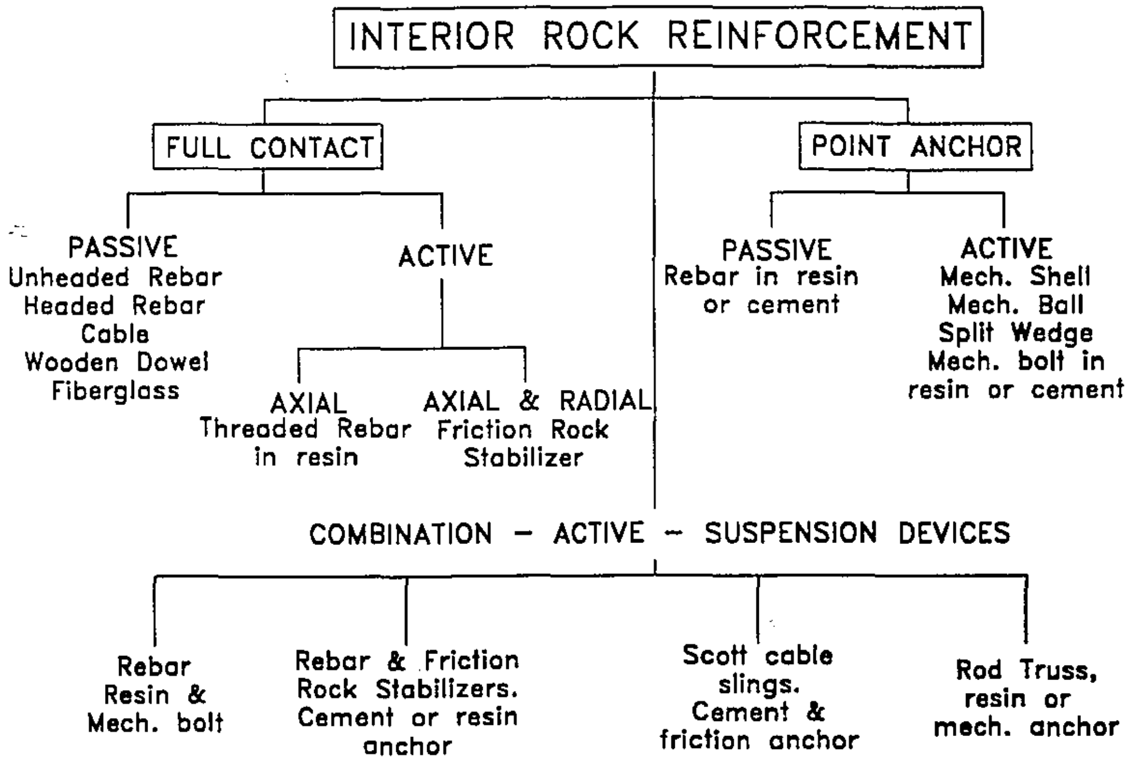


Figure 1.1: Classification of interior rock reinforcement (after Scott, 1987)

1.2 General Overview of Ground Support

Ground support is selected and designed according to the behaviour of mine rock structure. Mine can be divided into two general groups; Hardrock mines and Soft rock mines.

1.2.1 Hardrock Mines

In this group, the rock mass generally remains stable immediately and after some times of excavation. This is primarily attributed to the mechanical properties of hard rocks. The detachment and fall of loose rock blocks, however, is due to inherent discontinuities in the rock mass and mining induced activities. The long term effects such as excessive deformation of large span openings, and other structural instabilities (for example rockbursting) can occur in hardrock mines. For overall stability, rockbolting is applied to keep the loose rock blocks in place. Rockbolts (usually 1 to 2 meters long) are used to reinforce loose rock blocks present in the close vicinity of an excavated opening. Cable bolts, being flexible, are used to reinforce stope backs, hangingwalls and footwalls, etc., since they can be longer than rockbolts (8 - 30 m). Both rockbolts and cable bolts can be utilized as active or passive supports.

Backfilling stopes employs material such as de-slimed mill tailings, natural sand, cohesionless media, cementing agents, etc. Backfill imposes a constraint on the displacement of potentially loose portions of stope boundaries, and thus prevents the disintegration of the surrounding rock mass in low stress environments. Backfill helps to control the pseudo-continuous and rigid body displacement of stope-wall rock, induced by adjacent mining. Therefore, a properly confined backfill is viewed as an effective support element in the mine structure (Brady and Brown, 1985).

Shotcrete is another effective form of external support in hardrock mines. The use of shotcrete in civil engineering applications is well known. With the increased popularity of the New Austrian Tunnelling Method (NATM), the use of shotcrete grew rapidly, especially in Europe. Dry shotcrete with damp sand (fine aggregate) was shown to have the potential to work without creating dust relating problems in underground mines [Baz-Dresch, 1991]. A thin layer of shotcrete acts as a ductile support and creates a self-supporting arch with the rock mass, and hence prevents excessive deformation. The structural performance of shotcrete is largely dependent upon the shape of the opening; a circular opening gives better performance than a rectangular opening.

Cement or chemical injection grouting is a successful technique traditionally used to reinforce soils. This technique is also applied in a fractured rock mass to improve its cohesion. It is not considered efficient in terms of installation time and cost requirements.

1.2.2 Softrock Mines

This category of mines is further subdivided into two groups; (a) evaporite mines and (b) coal mines:

(a) Evaporite Mines

Structural instabilities in evaporite mines are generally caused by excessive creep or long term time dependent deformations around rock excavations. They may result in separation and failure of stratified rock in the immediate roof or the opening. Rockbolting, usually 2 to 3 meters long, is used as local support elements for both short and long term effects. Also, backfill has been recently introduced in salt mines as an environmental solution to mine waste. This should also significantly reduce the hazards caused by ground subsidence at the surface.

Wooden packs or chocks are normally built with hardwood or softwood elements, generally in lengths of 600 mm (2 ft), 750 mm (2 ft 6 in.) or 900 mm (3 ft) with a square section of 100 mm (4 in.), 125 mm (5 in.) or 150 mm (6 in.). Wooden packs are built on firm and clear foundations. Each pair of timbers is placed at right angles to those below in order to transmit the load directly and evenly between layers. Wooden support is built tight to the roof to make it active support as soon as the roof deforms downward (Underground Support Systems, 1979). Hydraulic chocks are a form of powered support that are occasionally used in potash mines. These are used at the face ends of roadheads; in anchor stations for face and gate conveyors; in development drivage; and, generally, to supplement other forms of supports.

Steel arches are widely used to support roadways in soft rock mines and they are often required to sustain large deformation. Steel sets or arches provide support rather than reinforcement. They can not be pre-loaded (active support) against the rock face and their efficiency largely depends on the quality of the blocking provided to transmit loads from the rock to the arches. Large deformation in steel arches can be accommodated by using special clamping elements designed to slip when predetermined loads are attained.

(b) Coal Mines

Coal mines are soft rock mines, however, their ground behaviour is generally different from evaporite mines. Supports are often provided to control local failure and to prevent excessive dilatation of discontinuities. The following are the most common types of support utilized in coal mines:

- Roofbolts and wooden dowels

- Wooden packs
- Powered supports or hydraulic props
- Steel arches

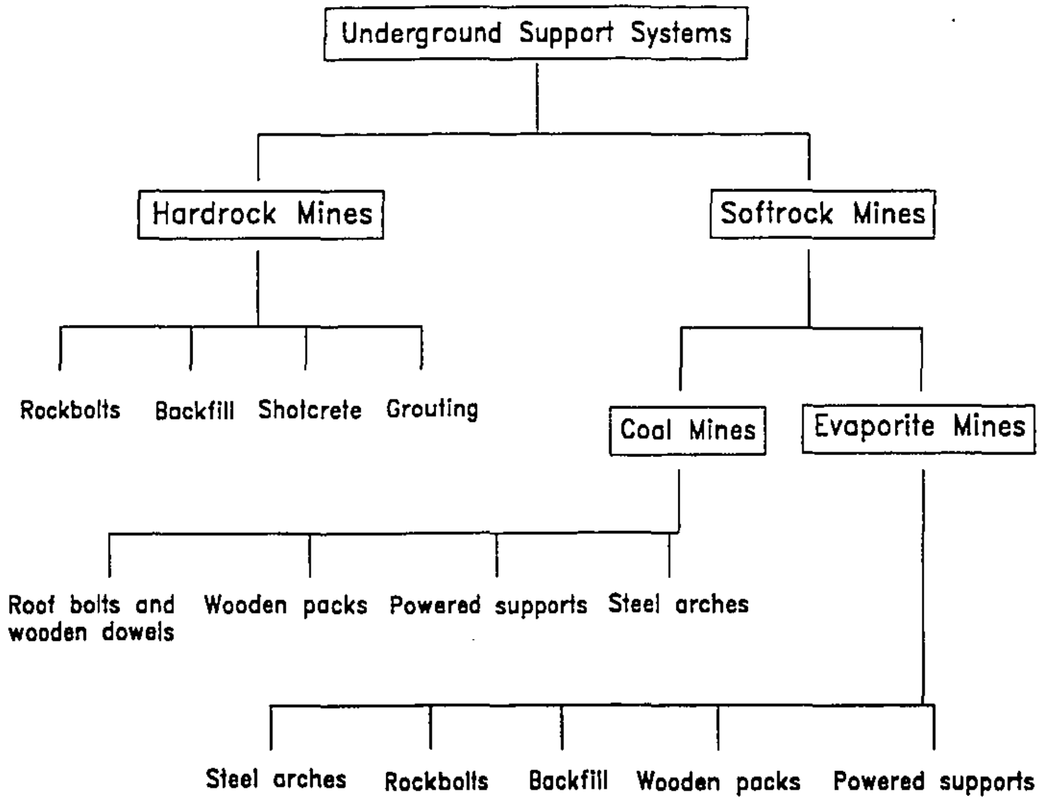


Figure 1.2: Commonly used underground support systems

1.3 Composite Material Ground Supports

With the advent of more efficient and automated mining such as continuous mining, the need for a compatible support system for such type of mining has become inevitable. Conventional steel support systems posed the following problems:

1. Inability to be cut by standard cutting bits.

2. Non-durable in a corrosive atmosphere such as wet, acidic ground.
3. Generate sparks when in contact with cutting tools.

Besides these three serious problems, conventional supports are heavy; which makes handling difficult. Their mechanical properties may not be altered easily to meet the specific requirement of certain ground conditions, for example, flexible reinforcing elements in a rockburst-prone area, very rigid support in a fractured area, etc.

A detailed research was initiated to identify composite material support applicable to both hard and soft rock mines. The targeted mining applications were mostly Canadian hard rock mines such as metal mines and soft rock mines such as potash mines, therefore, all efforts were focused on ground reinforcing tendons which are primarily used in these mines. The properties of composite materials are discussed in the following paragraphs:

With the concept of advanced mining such as continuous mining, it may often be necessary to mine through areas previously supported. This means that the cutting heads of excavating machine have to come in contact with supporting elements. Thus non-cuttable supports can jeopardize mining process. With conventional support, this problem can only be avoided either by using special steel cutting heads or by removing existing steel supports prior to machine advance. The former option is costly and the latter is costly as well as dangerous, since it may cause ground instability. A viable option for these two serious problems is a "cuttable" ground reinforcing element.

The cost of installation of support elements is often far higher than the cost of materials and equipment itself. A time consuming installation, which delays the mining process and lets the unsupported ground relax, is not acceptable to an efficient mining operation. Additionally, cuttable supports which are compatible with the existing installation equipment and methods are more desirable, since they will no longer require new installation equipment and training of mine personnel.

The property of supports being light weight is important when support installation becomes a part of continuous mining machines. The lighter the support elements, the less dead weight the machine has to carry, therefore, the machine efficiency would be increased. It would, as a result, decrease the delays which happen due to manual setting and removing of conventional steel support. The transportation of lighter support elements will also be easy and will reduce overall cost.

During the process of cutting composite materials with steel cutting tools, no sparks are likely to be generated. This property is important in the mines where there is a possible presence of highly flammable gases, for example methane in coal mines.

The life-span of different components of a mine such as haulage drifts, ore passes and ramps varies. Support should be durable enough to withstand and function properly during the working life of an opening. Moreover, support systems should also be durable against the occasionally corrosive environment. These environments could be due to the presence of moisture, acids, chlorides etc.

The existing design tools should equally be exercised on the new cuttable supports. This is possible if composite material supports are comparable to the conventional support systems in terms of mechanical and physical properties. This will not only ensure the acceptance of 'new' cuttable supports over conventional supports but also increase the factor of economy.

Although the factor of economy is interrelated with other factors such as cutter costs in continuous mining machines, installation process, storage, transportation, it is reasonable to expect that the initial investment of cuttable supports should not be excessive. An expensive support element can discourage any initial trials and, prevent further adaptation to the mining environment.

In summary, composite materials have certain advantages over steel. These are:

- Cuttability
- Ease of installation
- Ease of handling (light weight)
- Not to generate sparks during cutting
- Durability
- Comparable performance with existing supports
- Economy

1.4 Applications of Composite Materials

Composite materials have several potentially significant applications in mining. They have already been used in milling processes and in conveyer belts. Some of these applications in mining are:

1.4.1 Continuous Mining

Technological developments from 1950 to 1990 have resulted in a tremendous increase of productivity of coal mines in the United States. These developments are attributed primarily to the advancement in continuous mining techniques for longwall excavation and high-capacity-powered roof supports (Barczak, 1992).

In hardrock mining, drill and blast methods have been used successfully, however the method can be said to have reached its plateau of performance; and this is primarily because of its cyclic nature of operation. Therefore, continuous mining methods are under investigation by more and more researchers. Continuous mining machines have already been introduced in moderately hard rock mines. The discouraging factor, which is also a persisting problem, is the unavailability of compatible and suitable supports for the continuous mining process.

The first continuous mining machine, in hardrock mines, was developed and introduced in 1984 at Mount Isa Mines, Queensland, Australia. It was reported that over 10 % of delays was caused by support systems (Boyed, 1987). Similarly, in another survey by Terezopoulos (1987), it was reported that the delay of the drivage operation was mainly due to the setting of supports. He wrote, "An analysis of the time spent in a tunnelling operation showed that the machine was utilized for 30 % of actual cutting operation, 40 % for support setting and machine stoppages for external reasons, 30 %".

The concept of advanced continuous mining means continuous excavation, mucking and support. Therefore, support systems which can be adjusted behind continuous excavating machines are required. This is to ensure that supports are installed continuously as well as immediately after excavation, so that no time is to be wasted in halting the excavating operation for the sake of installation of supports. One important requirement to make the process successful is that the support element must be cuttable. Therefore, the use of any form of internal and external supports from such materials as steel is not favoured.

Types of Supports for Continuous Mining

A review of current continuous support systems and a survey of manufacturers and suppliers of supports, as well as the producers of new materials suggests that the new concept of continuous support has plenty of potential. It can be based on either pre-excavation or post excavation support systems (Hassani and Khan, 1990). A classification of supports for continuous mining is shown in Figure 1.3.

(a) Post Excavation Continuous Support Systems

In underground mining, support is basically required in post excavation rather than pre excavation situations. One basic purpose is to create a self-contained support system within the rock mass. The following is a classification of cuttable support for post excavation installation:

Cuttable Tendons

Cuttable rockbolts or cable bolts are found to be the most suitable support systems for continuous mining. These bolts are comparable to a great extent with conventional tendons both in mechanical and physical properties. The properties of composite tendons can be modified during their manufacturing process. Cuttable tendons are economical and efficient (due to their light weight) supporting elements. The main constituents of cuttable tendons from the manufacturing point of view are glass fiber, aramid and carbon, etc. and bonding matrices such as polyester resins, epoxy resins etc. The tensile strength of cuttable tendons is comparable with conventional steel rockbolts. The cost, however, varies to a large extent. Rockbolts and cable bolts made of composite materials offer significant potential for advances in continuous hard and soft rock mining technology.

Composite Drill Rod - Reinforcing Element (Drill and Support)

This concept offers good potential in multiple pass cutting operations (stope excavations). Extensive research and development is required to evaluate the governing parameters and costs involved. From the literature review, it is found that composite materials made of carbon and boron have tensile, compression and shear strengths which could enable a composite rod to act both as a drill rod and a rockbolts. Presently this technology is limited to the cutting tools and precious stone industry only.

Wooden Dowels and Bolts

Wooden dowels have been used in coal mines as cuttable rockbolts. However, due to insufficient support capacity and potential danger of fire, wooden dowels are, generally, not recommended in underground mining.

Shotcrete

This is an effective support system, however, its use is, generally, limited to soft rock mines. Additionally, it is an expensive and time consuming process.

Shotcrete and Composite Bolts

A combination of shotcrete and composite bolts is an ideal support system. This can provide immediate support behind the continuous mining machine, however, it is an expensive option.

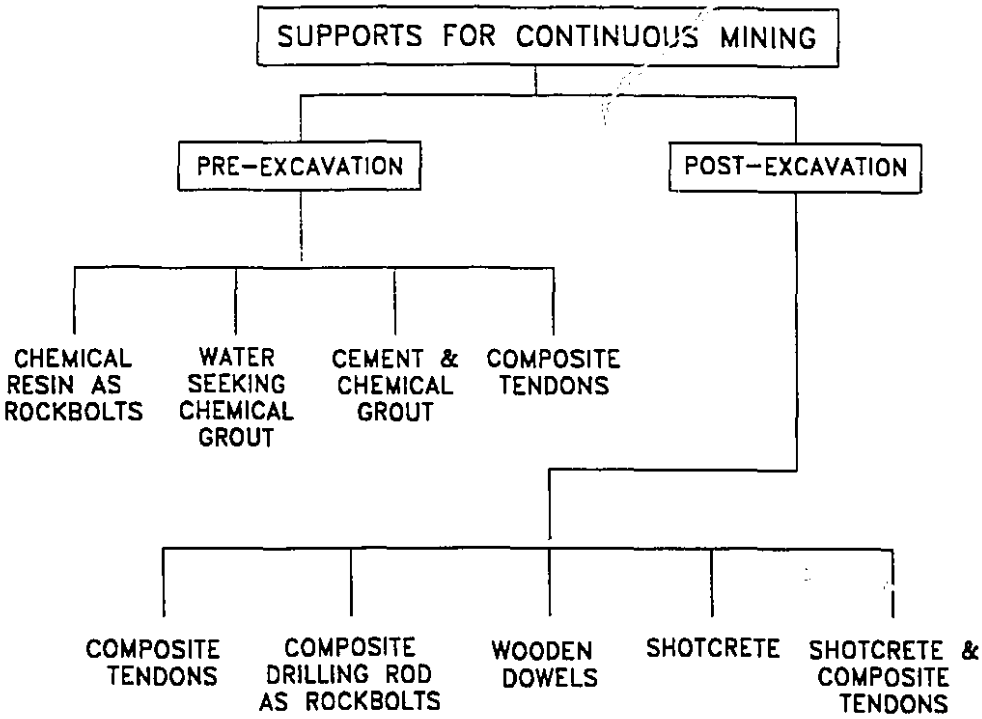


Figure 1.3: Classification of continuous mining supports

(b) Pre Excavation Continuous Support Systems

In a highly fractured rock mass, a stable excavation is seldom possible. Ground has to be reinforced prior to excavation. A number of options have been studied. These are briefly described below.

Chemical Resin Injection into Boreholes Acting as Bolts

Chemical resins and epoxies are used to repair small and controlled jobs. Using the same concept, the US Bureau of Mines (USBM) conducted a thorough research into commercially available "bonding materials" (Raff et al., 1973). No such bonding material has been found in their research which can stabilize a cracked opening in hard or soft rock mining. Epoxy and Polyurethane are two materials that received attention because of their extraordinary bonding strength. Epoxy is an expensive product and is not economically viable for large volume applications. In addition, the bonding layer of epoxy between two surfaces should not be more than 0.5 mm, otherwise a thick layer would (Raff et al., 1973).

The use of Polyurethane in underground mining environments might require very high pressure which could cause fracturing of the stable ground. Polyurethane can be successfully injected (cannot be sprayed) into discontinuities as a pre-excavation support method. But the use of Polyurethane is hazardous since lethal gases evolve during injection operation (Peruchon, 1989). Its cost is also high, ranging from $ 45.0 to $ 75.0 per gallon.

Special Water-Seeking Chemical Grout

The concept is that special water-seeking chemicals are injected into water filled boreholes drilled in the rock mass. This will not only produce a support element in the rock mass similar to cable or rockbolts, but also it would penetrate into discontinuities and bond them together. It would also act as a seal and prevent ground water seepage into the stope area. This method could be successful in highly controlled civil engineering applications, but in underground mining there is no such chemical that could provide sufficient bonding strength between rock discontinuities.

Grouting

Cement or chemical injection grouting can be used as cuttable reinforcing support in underground and surface mining. Additionally, this technique can be applied as post-excavation support. This is a costly technique and is used only for specific jobs such as in a fractured rock mass to increase its cohesion in order to consolidate the rocks.

Pinning by Composite Long Tendons

Since composite tendons can be flexible (many can be coiled in a 1-2 m diameter) and light weight, long tendons can be effectively used as fully grouted support in pre excavation support systems. A high strength cement grout (with adequate quantity of admixtures to increase the penetration of grout into crack openings) can be used: 1) to provide bonding between rockbolt/grout/rock and 2) to penetrate into discontinuities to further strengthen the overall rock mass.

Additionally, composite materials have the capacity to replace heavy steel structures like H or I section beams and columns. Composites can also be fabricated into rigid arches (curved H-sections), yieldable arches, posts and linear plates for shafts. Composites qualify to be used as ore pass liners to replace timber, steel, concrete or the various combinations of these materials currently used.

1.5 Justification and New Contributions

The justification for the introduction and laboratory evaluation of composite material tendons as a support system is that they offer the following advantages over conventional interior steel supports:

- Composite materials are cuttable; this property will most likely help to increase the efficiency of continuous mining machines. Which otherwise, need to stop for the manual removal of steel supports. At the same time, cuttable supports will also reduce safety factor, since removal of conventional supports can cause instabilities in the supported areas.

- Composite materials are not likely to corrode or degrade as an interior support in mining.
- Composite materials do not generate sparks during cutting. This will help to prevent fire hazards in underground coal mining.

The following is a brief review of the new contributions made through this doctoral research program:

1. Identification of composite material tendons compatible with mining constraints, found in the commercial and literature survey.
2. Laboratory investigation of different types of grouting materials such as high strength cement grout, shotcrete grout with high strength and conventional type 10 cements.
3. A detailed investigation of the design parameters for conventional steel cable. A section is devoted to the shear resisting capacity of fully grouted steel cable under different joint spacing.
4. A laboratory investigation of manually mixable polyester resin grout.
5. Laboratory evaluation of the interface characteristics of Arapree and Weldgrip tendons. The effect of different materials, mentioned above in 2 and 4, on the performance of the pull out capacity of tendons is evaluated.
6. Shear capacity of Arapree and Weldgrip tendons in variety of joint spacing is also investigated in laboratory testing.

1.6 Objectives

The main objective of this doctoral thesis is to investigate the mechanical performance of composite support elements for underground mining applications. Rigid and flexible tendons, which could replace conventional steel rockbolts and cable bolts, have been chosen as the focus of this research. To achieve this goal, the work was carried out in the following sequence:

1. Market and literature survey of composite materials.
2. Selection of commercially available composite tendons with the mechanical and physical properties that can be comparable to conventional steel rockbolts and cable bolts.
3. Experimental investigation of key parameters governing the behaviour of conventional steel cable bolts. Different types of grouts are also tested in this stage.
4. Preliminary experimental investigation on selected composite tendons to identify those with best potential.
5. Detailed laboratory investigation of those composite tendons identified in step 4.

1.6 Methodology

To evaluate the performance of composite tendons in a variety of ground conditions, the following properties of tendons are of prime importance:

1. Uniaxial tensile strength
2. Shear strength
3. Young's modulus of elasticity in axial direction under tensile load

4. Young's modulus of elasticity in transverse direction under compressive load
5. Pattern of surface profile and its strength
6. Bolt-grout interface strength
7. Shear strength of fully grouted bolt
8. Poisson's ratio

In the present research, it is required to experimentally evaluate items number 5, 6 and 7, to achieve the objectives mentioned in section 1.5. Whilst, the rest of the properties are provided by manufacturers of composite tendons, hence, these are not included in the research program.

Composite tendons are classified into rigid bolts and flexible cables. This classification is derived on the basis of the potential applications of bolts and cables in underground and surface mines.

Composite rigid bolts have the potential to replace the conventional rigid steel bolts; presently used in mining applications. The load carrying capacity of fully grouted rigid bolts primarily depends in the elastic region i.e. pre-slip or pre-failure region. If the bolts break or its interface fails, the load sustaining capacity becomes almost zero. Therefore, the performance of rigid bolts can be evaluated with simple pull out tests. The effect of insitu and mining induced stresses and stiffness of host rock mass does not play a very vital role on their overall performance.

Composite flexible tendons or cables are introduced to function as conventional 7-wire steel cable does in underground and surface mining. Cable can be used, as short as rigid bolts to reinforce local ground, and as long as 20-40 meters, to control excessive dilution of inherent and mining induced discontinuities in a stope under development. The functioning of cables is much more complicated and difficult than ordinary rockbolts. Minor changes in ground conditions such as insitu and mining induced stresses, mechanical properties of rocks, etc. can effect its performance. Therefore, a detailed research program was undertaken to evaluate all those parameters which can help to understand the behaviour of fully grouted cable. Because of unavailability of proper gripping system (or chuck) of composite cables and a large quantity of composite cables, it was decided to use conventional steel cable for evaluation of parameters. It is also believed that composite flexible cable, specially Arapree, and steel cable behaves identically, therefore, the results of steel cable can be generalized for composite cables.

In order to further confirm such hypothesis, a detailed laboratory experimental program was conducted on composite flexible cables in both cement and polyester grouts. The complete step by step research program is shown in Figure 1.4

1.7 Thesis Outline

This thesis consists of ten chapters. These are described briefly as follows:

- Chapter 1 describes the purpose of this research. It contains definition of internal and external supports, continuous mining supports and their characteristics and types of supports for continuous mining. Post and pre excavations continuous support systems such as cuttable tendons, wooden dowels, shotcrete, etc., are also described. The thesis objectives and methodology are also presented in this chapter.

- Chapter 2 contains the literature review of rockbolts, cable bolts and their mechanical behaviour. In the first section, the background history of rockbolts in underground mining is presented. The mechanisms of rockbolts as supporting elements, types of

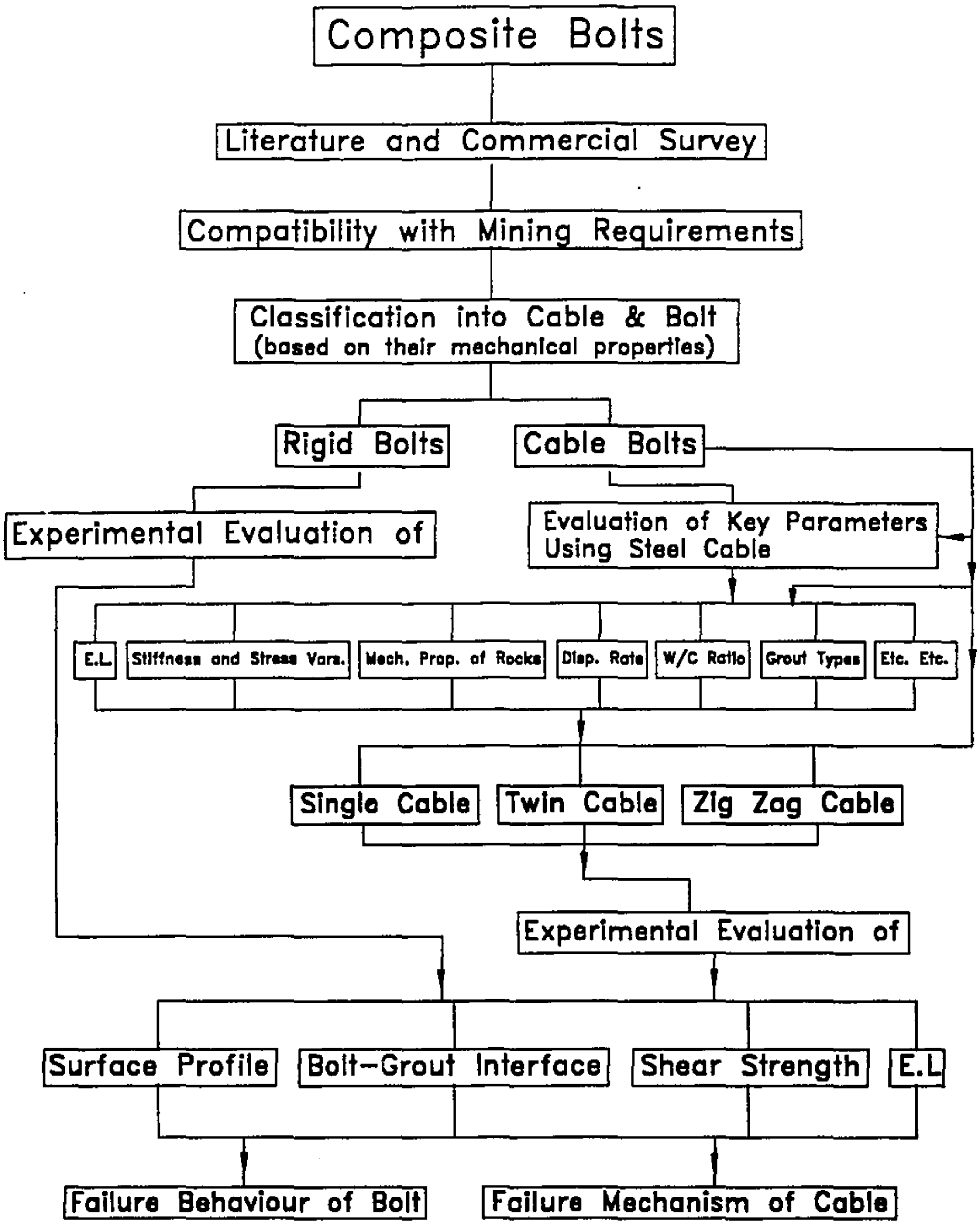


Figure 1.4: Flow chart of investigation of composite tendons

rockbolts and load bearing capacity of rockbolts are discussed. The second section gives a comprehensive literature search of all relevant information on cable bolting. The performance of fully grouted cable bolts, their failure modes and principal applications of cable bolts in underground mining are highlighted. In the third section, a comprehensive literature review on mechanical behaviour of fully grouted bolts are presented.

- Chapter 3 presents the laboratory experiments undertaken to investigate grouting materials. This contains testing of conventional grout, high strength cement grout, shotcrete grout and polyester resin grout.

- Chapter 4 goes in the investigation of the key parameters involved in the performance of fully grouted cable bolts. This contains laboratory experimental evaluation of parameters such as hole diameter of loading plate, effect of stiffness and stress variations, rate of displacement, water/cement ratio, cable embedment length, shear resisting capacity, etc. This chapter begins with the brief description of the experimental procedure and sample preparation. The results of each parameter are tabulated and presented graphically along with their interpretation for field applications and limitations.

- Chapter 5 aims to provide basic information on composite materials, their structure, types, advantages, environmental concerns and applications in industry particularly in mining.

- Chapter 6 introduces the commercially available patented composite tendons, which can be used as uniaxial tensile bearing elements. Sixteen different types of composite tendons such as Arapree, Weldgrip, Weidmann, Celtite, etc., are explained along with their physical and mechanical properties and their potential applications in mining industry.

- Chapter 7 presents an analysis of the pull out test response of commercially available composite tendons. It, in general, considers the effect of grout-bolt interface on their overall support behaviour. Based on the findings of this chapter, a thorough investigation of "acceptable" tendons were conducted and their results are presented in the following chapters.

- Chapter 8 explains the detailed laboratory investigation of Arapree flexible composite tendons. The effect of different types of grouts such as conventional cement grout, high strength cement grout, shotceret grouts and polyester resin grout were investigated to evaluate the pull out behaviour of the Arapree tendon. Different configurations such as double twin parallel Arapree tendon and twin zig-zag Arapree tendon were made in the laboratory, tested and their results are presented in this chapter. Experimental results show that Arapree could be successfully used with cement grout but not with the polyester resin grout.

- Chapter 9 goes in the detailed investigation of Weldgrip fiberglass rigid bolts with different types of grout. Experimental results show that Weldgrip tendons have the potential to be used as rockbolts with both cement and polyester resin grouts. The performance of hollow and solid bolts were found identical in pull out tests, however, the former resisted more shear load because of their higher flexibility than latter.

- Chapter 10 presents the conclusions and recommendations for future research.

Chapter 2

Literature Review

2.1 Rockbolts

In North America, the first attempt with rockbolts was made by Welsh Slate Quarry in 1872. In 1920, the St. Joe Company used Split wedge rock anchors to support surpass in the Old lead belt of Missouri. By the 1940's, they were employed in Australia (Elrington Colliery) and Norway (A/S Sulitjelma) (Ames & Graham, 1985). Strata bolting was introduced on a large scale in U.S. mines in late 1940's. The serious trials were started in 1951 with the concept that bolts should be installed as soon as possible after the roof is exposed and that they must be tensioned at installation (Adock & Wright, 1958). The early popularity of rockbolts, around the world, can be seen by the fact that rockbolts were first introduced, in early 1950's in France, to attach timber struts to the roof across the roadway axis. Struts were, however, replaced by simple end-plate, and as a result, consumption rose from 600,000 bolts in 1953 to 2.1 million in 1956. This number increased to 4.3 million with the introduction of mechanical bolting in 1960 (Dejean et al., 1983).

The United States Bureau of Mines found mechanical shell or bale anchored roof bolts as economical and efficient support elements in coal mine environment and therefore, in late 1940s' a research program was initiated to calibrate roof bolts. By 1960, roof bolts became important as well as economical and safe supporting elements (Scott, 1983). In 1970, U.S. Coal Mine Health and Safety Act enforced the law that underground openings must be supported and must not be operated under unsupported areas. A rapid increase in the use of rock bolts was resulted with the implementation of this act (Bolstad et al., 1983). In addition to such act, the introduction of many low-profile mining machines, in the late sixties, led to fast and safe installation of steel rock bolts and, consequently, increased their consumption in both hard and soft rock mining. Mechanical bolts or point anchors were introduced in mining at the same time. Fully grouted nontensioned bolts were introduced in early 1970's in poor grounds where mechanical bolts were not effective. Cement and resin grouts were used as bonding materials between the bolt and host rock. Friction Rock Stabilizer Rockbolt (Split Set) was introduced in 1977. Because of its rapid installation, by 1980, over three million fixers were used per year in metal and non-metal mines of United States (Scott, 1983). At the same time, Atlas Copco Co. developed Swellex bolt, which also got enormous popularity within the first few years of its introduction. The popularity of fully grouted bolts never reduced and, today, they constitute about half of the domestic use of bolts in the United States.

In Canadian mines, rockbolts are the basic supporting tool to stabilize weak strata around openings. The annual consumption for underground mines in Quebec has been estimated to be about 750,000 bolts in 1984 of which most (72 %) are mechanically anchored rockbolts. Cement and resin grouted rockbolts are gaining popularity and now account for about 18 % of total consumption. Friction bolts such as Split Set and Swellex account for 10 % of total (Choquet, 1991).

2.1.1 Supporting Mechanism

Pank (1956 and 1962) suggested that point anchor (mechanical) bolts support the mine roof effectively. It was concluded that for beam action to develop the bolts must be in tension. The tension in the bolts would produce a normal force between layers and thus the frictional forces can carry the horizontal shear stress. This idea was later rejected by various researches, the reason is that the tension in bolts can cause failure in the rocks that are located below the intact rocks. Fairhurst and Singh (1973) presented the mechanics of the support action of nontensioned fully grouted roof bolts. Their theory explains that beam action can be accomplished without tension in bolts and the horizontal shear can be carried by the shear stiffness of the grout/bolt combination. Presently, it is accepted that rockbolts fundamentally work in four different mechanisms (Snyder, 1983 and Habenicht, 1983). A brief explanation is given below:

Suspension Effect

Individual or limited rock blocks can be detached from the rock mass under its own weight and fall down causing a wedge failure. Rockbolts are designed to carry the weight of blocks. The bolt transfers the weight of the potentially failed rock to the intact rock (see Fig. 2.1 (a)). The probability of shapes, volume/weight and the sliding/falling direction of loose rock has been described in detail by Hoek and Brown (1980).

Nailing Effect

The nailing effect increases the magnitude of normal forces perpendicular to the joints. This in return increases the frictional resistance between the broken rocks and discontinuities. By this process, the fragmented or jointed rocks in the roof layer are stabilized, as shown in Figure 2.1 (b).

Beam or Slab Effect

In horizontally bedded sedimentary rocks, the roof is controlled by the strength of the bedding plane. The rock structure around an excavation in a sedimentary rock resembles an excavation in jointly bedded slabs or beams. Therefore, these beams or slabs undergo large deformation at the middle span and excessive shear stresses near the horizontal support. To control the failure, rockbolts are inserted to transfer the load of the unstable layers of rock to the stable solid rock. This is shown in the Figure 2.1 (c).

Arch Building Effect

If an opening with a curved roof is excavated in a jointed rock mass (mosaic), a natural arch of compressive stresses is formed at some depth into the rocks. This arch is primarily subjected to compressive stresses. The stability of this arch depends on its shape, type of ground and the stiffness of the arch abutments. Rockbolts are commonly used to reinforce both ceiling and arch abutments. In other words, rock bolts simply provide the constraints at the potentially slipping zones and consequently structures support itself (see Figure 2.1 (d)).

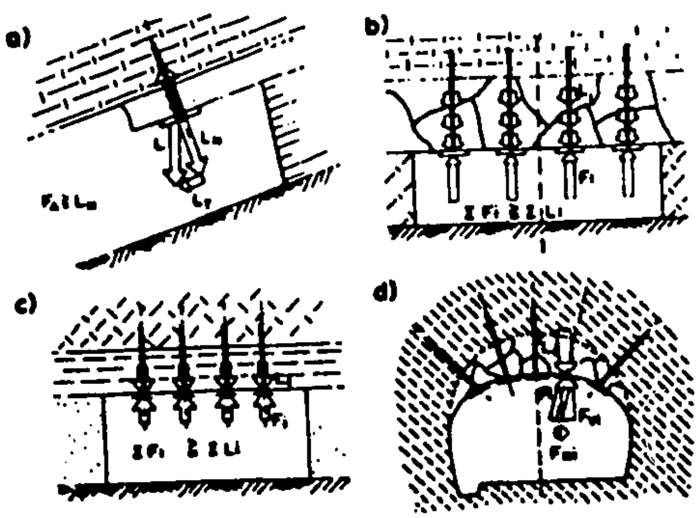


Figure 2.1: The mechanism of rockbolt support element (a) Slab in the roof, (b) Fragmented or jointed rock in a roof layer, (c) Roof stratum of low strength, (d) Fragmented zone above a curved contour (Habenicht, 1983)

2.1.2 Types of bolts

There are generally three types of rockbolts which are presently used in the mining industry. These are:

1. Grouted Bolts
2. Mechanical Bolts
3. Frictional Bolts

Grouted Rockbolts

Grouted rockbolts have been widely used worldwide for last four decades in both civil and mining engineering applications. The most common types of such bolts are threaded steel rebars. With their unique surface profile and mechanical properties, these bolts are patented under various names such as Dywidag Steel. Cement and resin grouts are used as bonding agent. In civil engineering applications, fully or partially grouted rockbolts are commonly used as pretensioned bolts as permanent supports. In mining, fully grouted untensioned bolts are generally applied as temporary supports.

Cement and organic grouts are used to install fully grouted bolts. Both these grouts have slightly different installation techniques and a brief description is as follows:

Two methods of installation of nontensioned (cement) fully grouted bolts are currently in process. The first conventional method consists of injecting grout and filler into the borehole either before or after insertion of the rod. The second method is totally new; it employs prepacked cartridge containing cement, plaster, filler and the required amount of water, and are used in the same way as resin cartridges. The water is contained in 1 mm pierces the cartridge and mixes the contents, which set fast in several minutes. A tensile strength of approximately 8 tonnes is attained after 30 minutes. This method appears to be cheaper and, in particular, eliminates the problems of storage of the chemical grout (Bolstad et al., 1983). Figure 2.2 & 2.3 show two types of grouted rockbolts (rebar & Dywidag steel) along with their technical data. The advantages and disadvantages of bolts are also mentioned.

Mechanical Rockbolts

The expansion shell anchored rockbolts, of standard or bail type, are the most common form of mechanically anchored rockbolts. This type of rockbolts consists of two parts; (a) a long tendon or rod, and (b) a wedge shaped expansion shell anchored at the end of tendon. First the bolt is inserted into the borehole and then rotated. This rotation opens the expandable shell into conical wedge to anchor with the wall of the borehole. The applications of this bolt are both in mining and civil engineering. However, the applications are limited to moderate hard rock environment. In very hard rock and weak rocks, the anchored end does not work effectively. Figure 2.4 shows a mechanical rockbolt with technical data.

Frictional Rockbolts

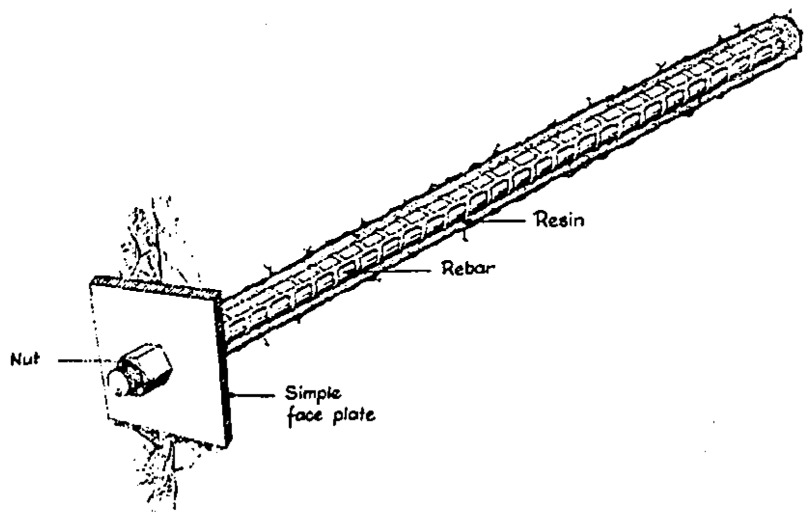
Swellex and Split Set are two common examples of frictional rockbolts. Their basic purpose is to provide an immediate support without getting affected by moisture contents in the rock. Full length frictional resistance under the radial force against the borehole wall is the basic supporting mechanism of this type of bolts to bond the broken rocks to the intact rocks (Scott, 1983). Swellex is a collapsed steel tube with sealed ends. The tube is hand inserted into the borehole. High pressure water is then used to expand it to the configuration of the borehole surface. Figure 2.5 shows Swellex and its technical data, advantages and disadvantages.

The Friction Rock Stabilizer or Split Set is a hollow steel bolt approximately 38 mm in diameter with a 12.7 mm slot along its entire length. One end of the bolt is tapered to facilitate its entry into the borehole and the other end has a welded ring flange to hold the roof plate. Split Set is driven by impact into the bore hole and this impact opens the slit along the entire length of the bolt. The bolt, thus, transfers the weight of loose rocks both axially and radially along its entire contact length. The drilled hole for the Split Set is kept 3 mm smaller than the expanded bolt to obtain a force-fitted contact between bolt and hole (Scott, 1980). Figure 2.6 shows Split Set and its technical data, advantages and disadvantages.

2.1.3 Design Methods

Traditionally, rockbolt support systems are designed based on one or more of the following methods.

- Analytical Methods
- Empirical Methods
- Numerical Methods

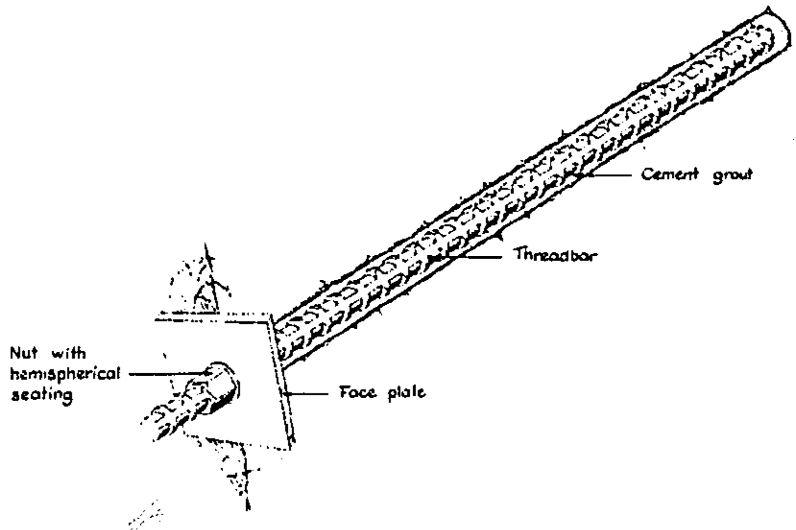


Steel quality designation	570 N/mm^2	83 kpsi
Steel diameter	20 mm	7/9 in
Yield load, steel:	120 kN	13 tons (US)
Ultimate load, steel	180 kN	20 tons (US)
Ultimate axial strain, steel	15 %	15 %
Weight of bolt w/o face plate and nut	2.6 kg/m	1.75 lb/ft
Bolt length	any length required	
Recommended borehole diameter	35±5 mm	1 3/8 in

Advantages
The bolt gives rapid support action after installation. If a "fast-setting" resin is used for bottom anchoring of the bar, the fully grouted rockbolt can be tensioned. High corrosion resistance in permanent installations.

Disadvantages
Difficulties with the resin cartridges in underground environment which can affect installation reliability. Resin can be messy and hazardous to handle as well as wasteful. Resin has limited shelf life.

Figure 2.2: Fully grouted "Rebar" rockbolt and its technical data, advantages and disadvantages (Stillborg, 1986)

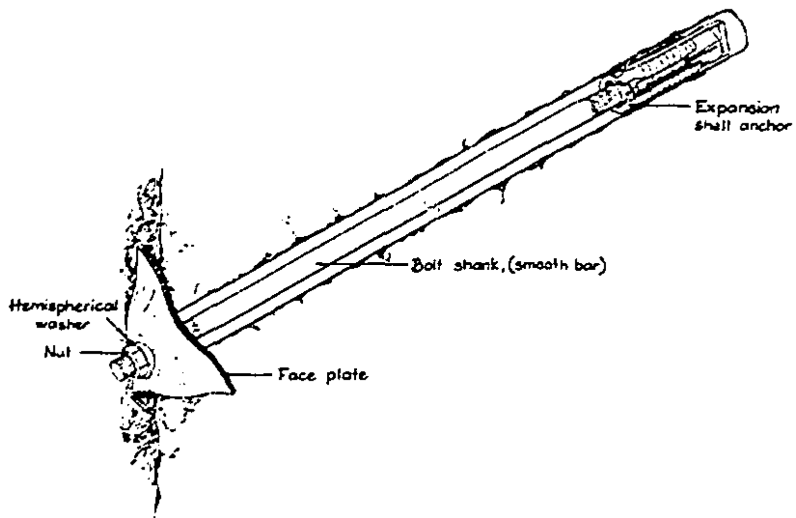


Steel quality designation	1080 N/mm^2	157 kpsi
Steel diameter	20 mm	7/9 in
Yield load, steel:	283 kN	31 tons (US)
Ultimate load, steel	339 kN	37.5 tons (US)
Ultimate axial strain, steel	9.5 %	9.5 %
Weight of bolt w/o face plate and nut	2.6 kg/m	1.75 lb/ft
Bolt length	any length required	
Recommended borehole diameter	35±5 mm	1 3/8 in

Advantages
Properly installed, it is a competent and durable reinforcement system. Effect of corrosive environment is minimal. The system gives high bolt loads in various rock conditions.

Disadvantages
Expensive. Tensioning of the rockbolt is possible only if special installation procedures are followed. Use of standard cement in the grout requires several days curing before the bolt can take load. Quality of grout is difficult to check and maintain constant.

Figure 2.3: Fully grouted "Dywidag" rockbolt and its technical data, advantages and disadvantages (Stillborg, 1986)

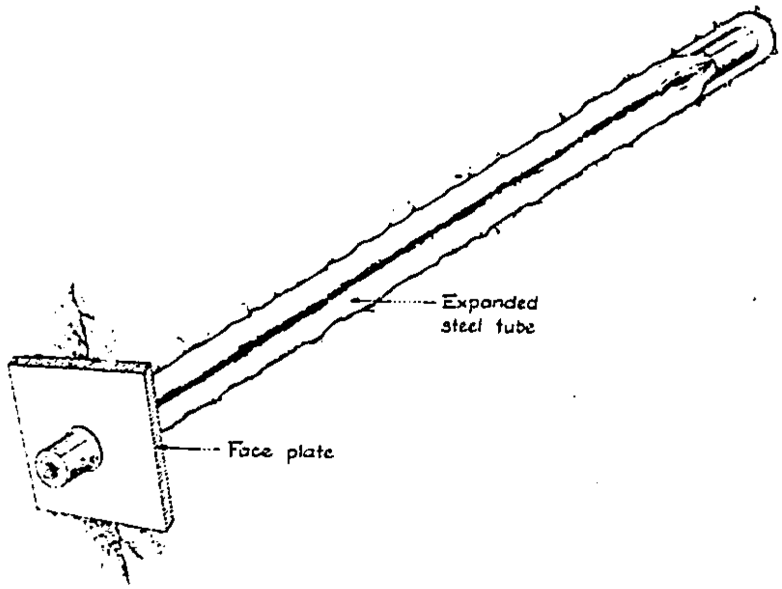


Steel quality designation	700 N/mm^2	102 kpsi
Steel diameter	16 mm	5/8 in
Yield load, steel:	140 kN	15.5 tons (US)
Ultimate load, steel	180 kN	20 tons (US)
Ultimate axial strain, steel	14 %	14 %
Weight of bolt w/o face plate and nut	2 kg/m	1.34 lb/ft
Bolt length	any length required	
Recommended borehole diameter	35 - 38 mm	1 3/8 in

Advantages
Relatively inexpensive. The bolt gives immediate support after installation. By rotating the bolt, a torque is applied to the bolt and tension accumulates in the bolt. By post-grouting, the bolt can serve as permanent reinforcement. In hard rock, high bolt loads can be achieved. It is a versatile system for rock reinforcement, assuming hard rock conditions.

Disadvantages
Limited to use in moderately hard to hard rock. Difficult to install reliably. Must be monitored and checked for roper tensioning. Loses bearing capacity as result of blast vibrations or when rock spalls off around borehole collar due to high rock stresses.

Figure 2.4: Mechanically anchored rockbolt and its technical data, advantages and disadvantages (Stillborg, 1986)

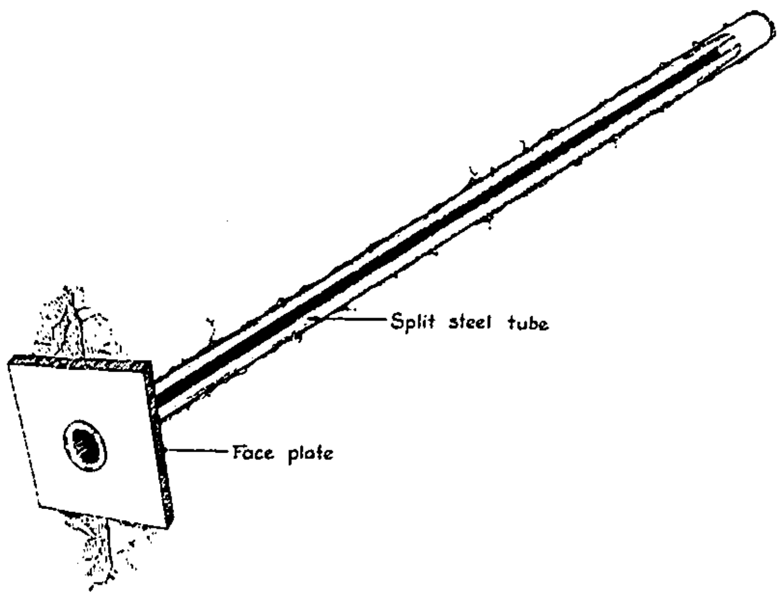


Tube diameter	26 mm	1 in
Yield load, steel tube:	130 kN	14 tons (US)
Ultimate load, steel tube	130 kN	14 tons (US)
Ultimate axial strain, steel tube	10 %	10 %
Weight of bolt w/o face plate	2 kg/m	1.34 lb/ft
Bolt length	any length required	
Recommended borehole diameter	35±3 mm	1 3/8 in

Advantages
Rapid and simple installation. Gives immediate support action after installation. Can be used in a variety of ground conditions. The installation causes contraction in the bolt length. This effectively tensions the face plate against the rock surface.

Disadvantages
Relatively expensive. Corrosion protection required if used in long term installations. Requires a pump for installation.

Figure 2.5: Swellex - a frictional rockbolt and its technical data, advantages and disadvantages (Stillborg, 1986)



Tube diameter	39 mm	1 1/2 in
Yield load, steel tube:	90 kN	10 tons (US)
Ultimate load, steel tube:	110 kN	12 tons (US)
Ultimate axial strain, steel tube	16 %	16 %
Weight of bolt w/o face plate	1.8 kg/m	1.2 lb/ft
Bolt length	any length required	
Recommended borehole diameter	35 - 38 mm	1 3/8 in

Advantages
Simple installation. Gives immediate support action after installation. No hardware other than a jackleg or jumbo boom for installation. Easy application of wire mesh.

Disadvantages
Relatively expensive. Borehole diameter is crucial in the prevention of failure during installation and in the provision of the intended holding force. Successful installation of longer bolts can be difficult. Cannot be used in long term installations unless protected against corrosion.

Figure 2.6: Split Set - a frictional rockbolt, its technical data, advantages and disadvantages (Stillborg, 1986)

Analytical Methods

The analytical methods currently available for the design of rockbolts are merely based on a limit equilibrium approach. Whereby the number of bolts is determined from consideration of the total weight of loose or potentially unstable rock and the ultimate strength of a single bolt.

In a situation where a rock wedge or a block is likely to fall down by gravity (see Figure 2.7), Stillborg (1986) recommends the following equation:

$$N = (W \times f)/B \qquad (2.1)$$

Where

N = number of rockbolts

W = weight of wedge, (density of rock approximately 2.7 t/m^3)

f = safety factor, usually $2 \leq f < 5$

B = load bearing capacity of the bolt.

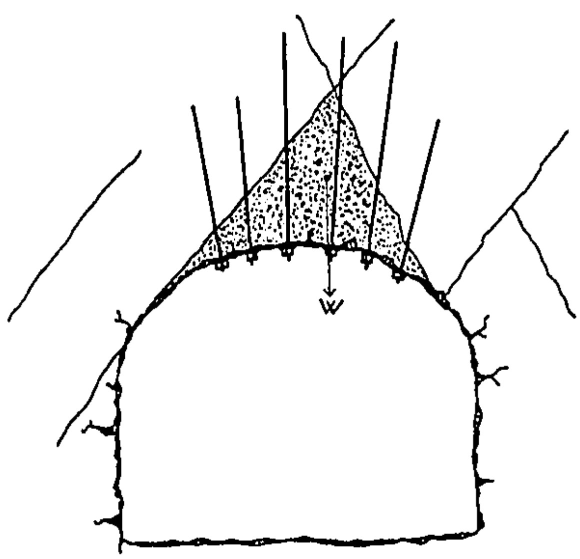


Figure 2.7: Reinforcement of loose wedge by bolts (Stillborg, 1986)

When a potentially loose block is likely to slide down before it is completely detached from the rock mass, the shear resistance offered along the surface of sliding should be included; see Figure 2.8. In this case the required numbers of bolts are calculated from (Stillborg, 1986)

$$N = \frac{W(f \sin\beta - \cos\beta \tan\phi) - cA}{B(\cos\alpha \tan\phi + f \sin\alpha)} \tag{2.2}$$

Where:

N = number of rockbolts
W = weight of wedge, (density of rock approximately 2.7 t/m^3)
f = safety factor, usually $1.5 \leq f < 3$
β = dip of the sliding surface
ϕ = angle of friction of the sliding surface
α = angle between the plunge of the bolt and he normal to the sliding surface
c = cohesive strength of the sliding surface
A = base area of the sliding surface; in meters
B = load bearing capacity of the bolt

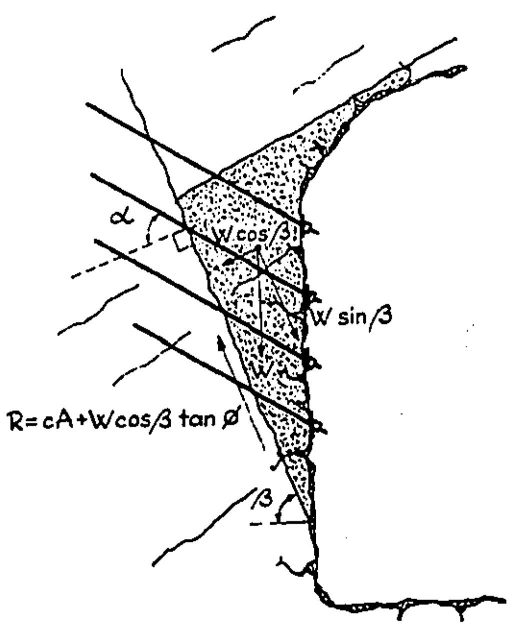


Figure 2.8: Reinforcement of wedge prone to slide under its own weight (Stillborg, 1986)

When stratified horizontal rocks are likely to fail in the middle of span due to excessive deformation and the depth of loose layers are in the range of 1 to 2 meters, rockbolts are then designed to transfer the weight of unstable layers to solid and intact layer (see Figure 2.9).

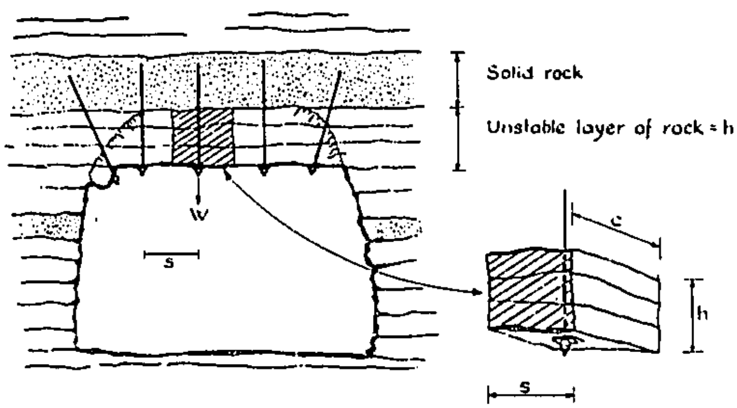


Figure 2.9: Transfer of weight of stratified rock into solid rock by means of rockbolts (Stillborg, 1986)

$$W = f \times s \times c \times h \times \rho \qquad (2.3)$$

Where

W= weight of rock to be supported by a single bolt; in tons

f = safety factor; (usually $1.5 \le f < 3$)

s = bolt spacing, perpendicular to axis of excavation; in meters

c = bolt spacing, along the axis of excavation; in meters

h = thickness of unstable layer or rock; in meters

ρ = rock density, approximately 2.5 t/m^3

If, however, the depth is more, then rockbolts are installed in order to convert thin slices of layers into one thick layer. This will decrease the deformation of slab and increase its stability.

The formation of natural arch at some depth in the roof above the ceiling of an opening in jointed rock mass e.g. mosaic rock, stabilizes the ground around the opening. The loose zone of rocks which form between arch and excavated boundary (see Figure 2.10), prone to fail by falling down under its own weight. Bolts are designed to suspend and/or stick loose zone to the solid and intact rocks (see Figure 2.10). This is achieved by calculating the effective length of bolt, using the following equation (Stillborg, 1986):

$$\text{Bolt length, } L = 1.4 + 1.84a \qquad (2.4)$$

Where 'a' is opening width; in meters

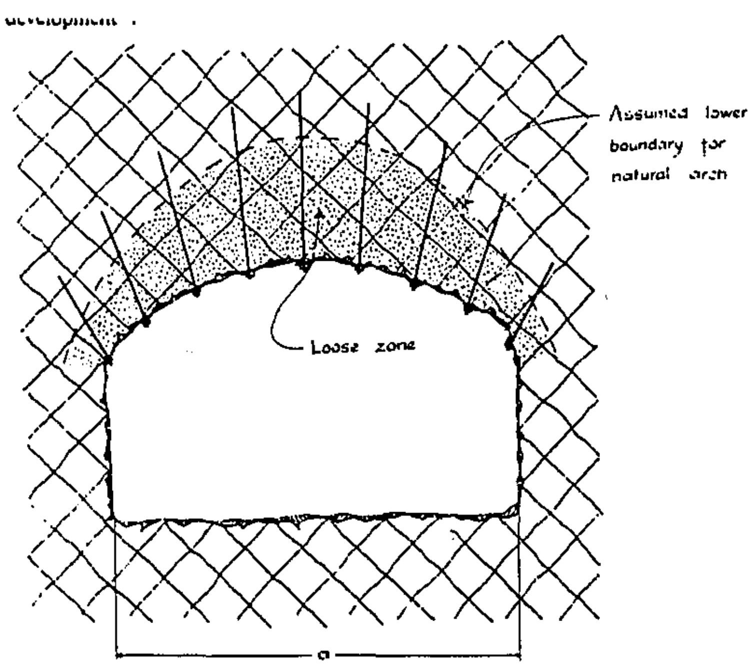


Figure 2.10: Transfer of the weight of loose zone above the naturally created arch (Stillborg, 1983)

Empirical Methods

The success of empirical methods depends on the selection of a proper method and the use of sound practical experience. The following are the geotechnical ground classification methods used to design the rockbolts:

a) CSIR classification (South African Council for Scientific and Industrial Research) developed by Bieniawski (1973).
b) The NGI classification (Norwegian Geotechnical Institute), developed by Barton, Lien, and Lunde (1974).
c) The Laubscher classification (1977).

These methods are documented in the literature, however, a brief explanation of the application of these methods in Canadian mines is given here. The need for rockbolts in a gallery or stope can be derived using the NGI classifications (Barton et at., 1974), and/or CSIR classification (Bieniawski, 1973). Both CSIR and NGI classification were modified by inserting a new line or symbol which shows the compatibility between the requirement of support systems in the Canadian shield and the existing classifications (Choquet, 1991). Primarily, these figures are used to decide whether to apply rockbolts or not. Barton et al. (1974) (NGI classification) proposed the following formula to estimate the spacing 'S' of the bolts:

$$S = C \times 10^{-3} / (P_{roof} \text{ or } P_{wall})^{-1/2} \tag{2.5}$$

where;

C = load exceeding the yield strength of the bolt, in kN

P_{roof} = Support pressure at roof

P_{wall} = Support pressure at wall

Table 2.1 shows how to calculate support pressure and corresponding length of the bolts. An empirical minimum rock bolting density, based on detailed survey of Quebec mines, was recommended for mines in Canadian Shield by Choquet (1991). Similarly, rules describing spacing and length of bolts on the basis of empirical methods are given by U.S. Corps of Engineers (Choquet, 1991) and Farmer and Shelton (1980). Table 2.2 shows parameters and governing empirical rules.

Table 2.1: Support pressure and length of the bolts, based on the NGI geomechanics classification (Barton et al., 1974)

Support pressure	Length of the bolts
$P_{roof} = (0.2/J_r).\ Q^{-1/3}$, if the number of discontinuity sets > 2	$L_{roof} = 2 + 0.15\ B/ESR$
$Proof = (0.2\ J_n^{1/2}\ .\ Q^{1/3})/3\ J_r$, if the number of discontinuity sets ≤ 2	
P_{wall}: Calculated with the same formulas as P_{roof}, by replacing Q by Q', with: $Q' = 5\ Q$ if $Q > 10$ $Q' = 2.5\ Q$ if $0.1 \le Q \le 10$ $Q' = Q$ if $Q < 0.1$	$L_{wall} = 2 + 0.15\ B/ESR$

Where:

J_r = Joint roughness number
J_n = Joint set number
Q, NGI = Geomechanical classification rating
ESR = Excavation support ratio
L = Length of the bolts (m)
P = Support pressure, in MPa
B = Span of the excavation (m)

Table 2.2: Empirical rules for minimum length and maximum spacing used for rock reinforcement (Choquet, 1991)

Parameter	Empirical Rules
Minimum length	Greatest of: A. Two times the bolt spacing B. Three times the width of critical and potentially unstable rock blocks* C. For elements above the springline: 1. Spans less than 6 m - 1/2 span 2. Spans from 18 m to 30 m - 1/4 span 3. Spans 6 m to 18 m - interpolate between 3 m and 4.5 m length, respectively D. For elements below the springline: 1. For openings less than 18 m high - use lengths as determined in C above 2. For openings greater than 18 m high - 1/5 the height
Maximum spacing	Least of: A. 1/2 the bolt length B. 1 - 1/2 the width of critical and potentially unstable rock blocks C. 1.8 m**
Minimum spacing	0.9 m to 1.2 m

* Where the joint spacing is close and the span is relatively large, the superposition of two bolting patterns may be appropriate; e.g., long heavy bolts on wide centers to support the span and shorter and thinner bolts on closer centers to stabilize the surface against raveling due to close jointing.

** Greater spacing than 1.8 m would make attachment of surface treatment such as chain link fabric difficult.

Numerical Methods

Numerical methods of analysis like finite elements, boundary elements and distinct elements have been used successfully for the simulation of rock excavations. The following is the list of popular software developed exclusively for mining involving rockbolts applications:

1) e-z tools (Mitri, 1993)
2) FLAC (Itasca, 1991)
3) U-DEC (Itasca, 1991)

2.1.4 Innovations in Rockbolts

Whenever, inspite of proper designing of support systems, a reinforced rock mass failed or existing support systems become uneconomical, researchers always try to come up with new but effective and economical solutions. Some of these ideas have already become reality while other are in the process of evaluation. The following is the description of such efforts in the field of rockbolts put together by various researchers:

Pumpable roof-bolt system was developed by the USBM. In a number of U.S. coal mines, corrosive atmospheres reduce the effectiveness of metallic bolts. In addition, some mining schemes benefit from cuttable supports. Both of these issues were addressed as the USBM was developing a pultruded fiberglass fully grouted bolt that can be installed to a length longer-than-seam height (Bolstad et al. 1983). The system consists of a hollow fiberglass-reinforced bolt and a pumpable adhesive such as resins, polyester, urethanes, acrylics, and inorganic adhesives. All were examined for applicability; however, most did not pump well because of resin instability. The high cost of urethane and acrylics, and low adhesive strengths of other, eliminated all materials except epoxies. A coal-base epoxy was developed which could be statically mixed and pumped by conventional equipment without additional heat or initial jet-setting (Soloman, et al., 1983). The bolt cores are rolls of fiberglass-reinforced polyester, each a 1/3-round section which can be easily bent at a bolt-to-roof placement head and formed at the bolt hole into a fully-round hollow bolt. This allows for longer-than-seam-height insertion. The material is reeled from separate rolls on the rear of a roof-bolter machine. As the bolt is formed, epoxy grout is extruded into the hole simultaneously, resulting in a fully grouted hollow core bolt. Resin mixing and pumping system components are kept clean by a simple water purge system. A bolt head or plate is not yet developed. The field evaluation of this system was planned for late 1993 by the USBM.

Dejean et al. (1983) suggested a rock bolt made of reinforced polyester or plastic laminated wood which can replace the existing steel bolts. The advantages of "new" bolts being non-corrosive and lacking in resilience, which makes this particularly suitable in case where the reinforced strata requires to be cut at some later date.

Ludvig (1979) came up with resin-grouted glass fiber bolts of the slack type as an alternative for supporting ore roofs. These bolts appeared to give quick support effect and are not likely to " contaminate " the blasted ore as steel bolts do.

2.2 Cable Bolt

The concept of fully grouted cable as active or passive support system was first introduced in mining engineering some thirty years ago. A cable (passive) is installed in the rock mass well before the stress relaxation and rock deformation. This is termed as pre-reinforcement of ground. The cable deforms in tension when a relative displacement occurs in the rocks surrounding the cable. The process of regaining an equilibrium of the excavated ground around a stope generates axial forces in the cable. In other words, the regain of equilibrium of disturbed ground due to stoping is controlled by cable support systems. In this process, the cable support system mobilizes the rock mass strength and transfers the load to the more competent ground. The presence of cables helps limit the amount of dilation and subsequent loosening of rock.

In certain mining applications, such as the reinforcement of cut and fill stopes and ore or waste passes, cable bolts have shown their unique support capabilities. The steel cable is flexible enough to be coiled and can be installed in relatively large lengths in underground excavations with very low headroom. Table 2.3 shows the technical data of steel cable commonly used to reinforce the ground.

2.2.1 Performance of Fully Grouted Cables

The performance of fully grouted flexible cable depends largely on the following three components of systems:

1. Ultimate tensile strength of tendon
2. Shear bond strength of grout
3. Stiffness of host medium (rock mass)

Table 2.3: Technical data of conventional seven wire steel cable (strand)

Total number of wires	7
Wire diameter	5.05 mm
Nominal cable diameter (measured)	15.2 mm
Calculated diameter of cable (Stheeman, 1982)	15.15 mm
Calculated perimeter (ditto)	63.34 mm
Cross section area	138.7 mm^2
Modulus of elasticity	194±6 GPa
Breaking load	270 kN
Poisson's ratio	0.3
Weight	1.1 kg/m

2.2.2 Failure modes

The most commonly observed failure mode is the failure of grout-tendon interface. According to Littlejohn and Bruce (1977) cable support systems may possibly fail in one the following modes (Figure 2.11 shows these modes) :

1. Failure within the rock mass
2. Failure of the tendon
3. Failure of the rock-grout interface
4. Failure of the grout-tendon interface.

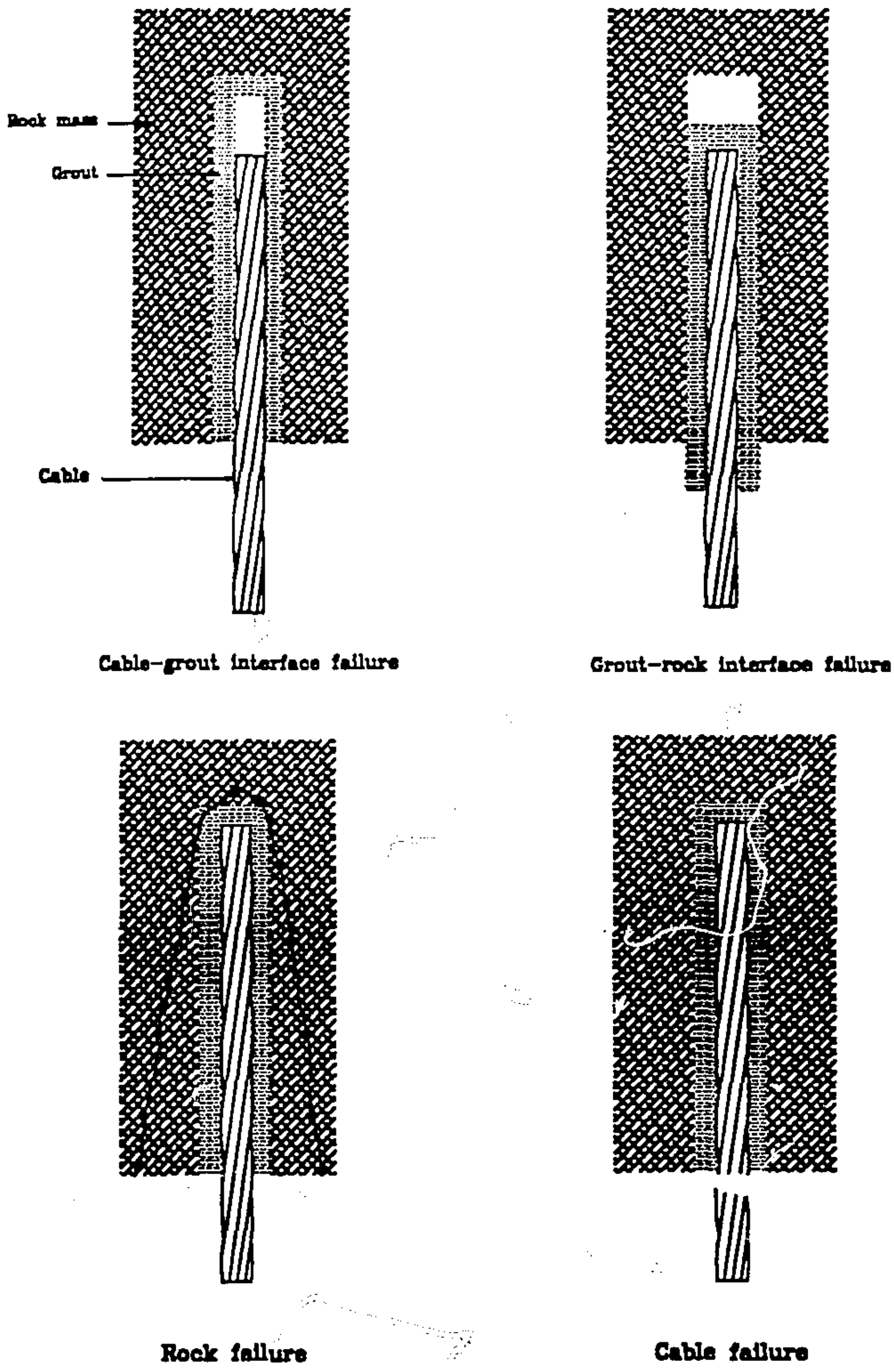


Figure 2.11: Possible failure modes of fully grouted cable bolt (Littlejohn and Bruce, 1977)

2.2.3 Development of Cable Bolt

The design and implementation of cables as supporting or reinforcing elements in surface and underground mining is complicated and requires detailed investigation of rock mass needs to be supported. Its applications are much more effective and innovative than barely replacing cable to stiff conventional steel bolt. The designing of rockbolts can be based on Geomechanics Rock Mass Classifications (RMR, Q-method, etc.) or one's personal experience. But design of flexible cable needs more understanding of the detail behaviour of rock mass and more sophisticated tools like numerical modelling techniques.

The use of cable bolts in mining is an analogous of cables being used to pre-reinforce concrete in prestressed concrete industry. However, the development of cable bolting as a support in mining is based on field and laboratory studies. The relative parameters and design guidelines have been generated in this field over the last three decades.

The earliest example of applications of cables as reinforcing element was found in Cheufes Dam, Algeria in 1943 (Littlejohn, 1980). In the mining industry, it is believed that cables were first used at the Willroy Mine in Canada (Marshall, 1963), and one year after cables were also applied at the Free State Geduld Mine Ltd. in South Africa (Thorn and Muller, 1964). According to Gramoli (1975) cable bolting was successfully applied at the Noranda Geco Division in Manitwedge, Ontario, to reinforce the backs and walls of large open stopes. This was the first typical cable bolting applications. By that time, cable bolts were formed of discarded wider rope and smooth pre-stressing wires (Windsor, 1992). Present day 7-wire strand was first used in the early 1970's in Broken Hill, Australia. Hunt and Askew (1977) applied cables, for the first time, as a systematic reinforcing support in cut and fill stopes in Australia.

Fuller and Cox (1975) performed experiments on cables comprised seven, 7 mm diameter high tensile steel wires, evenly spaced around a polythene or nylon air bleed tube. They concluded that because of smooth surface of the cable, the bond strength at grout-cable interface is very weak but in the pull out tests of (today's) 15.2 mm strand, they found that mechanical interlocking between twisted cable and grout were the main load transferring phenomenon. Chemical addition, however, plays a minor role. They compared the performance of clean and rusted wire and they found that latter offered more resistance.

Littlejohn and Bruce (1977) documented the possible failure modes of a fully grouted cable bolt; which are failure within the rockmass, failure of tendon and failure of either cable-grout interface or grout-rock interface.

Fuller (1981) instrumented and designed the length and spacing of fully grouted (cement + water) cables to support a cut and fill stope. These experiments also included the effect of blasting on the behaviour of reinforcing cable, and confirmed that cables are an effective reinforcing agent.

Stheeman (1982) carried out detail investigation of factors involved in the failure at Tsumeb Mine supported by cable bolts. He investigated the key parameters such as grout mixtures, bond stress at cable-grout and grout-rock interface, diameter and grade of steel, drill hole diameter, ground mass and effective length of cable bolt.

Goris and Conway (1987) included conventional strands (7-wire cable), epoxy-coated strands, conventional strands with steel buttons passed on them and birdcage strand in their research. This research concluded that epoxy coated strands resist more load than conventional strand but failure behaviour is similar. Moreover, they showed that with similar diameter, birdcage configuration increased the load carrying capacity of the cable, however, post-peak failure of birdcage cable is, in some cases, less than conventional strand. A summary of the development of cable bolt configuration is given in Figure 2.12.

Other researchers like Ortlepp (1983), Lappalainen at al. (1983), Bywater et al. (1983) evolved useful data from the observation made through monitoring the installed cable in the field i.e. cut and fill, open stopes, etc.

Over the period of three to four decades, several theories were modified because of availability of better tools for research. The concept of cable bolting to prevent occurring of fractures has been changed. The new approach is that the fracturing caused by excessive stresses are controlled by cable bolting, which increases the frictional resistance between the fractured rock blocks and, consequently, prevents excessive movement along and/or across joint (Lappalainen et al., 1983).

The other approach of installing tensioned cables is also replaced by untensioned cables. Apart from practical difficulties, the following are two main reasons:

a) Cables are installed prior to the excavation to pre-reinforce the ground. Immediately after excavation, the ground moves inward and as a result, the passive cables get stretched and become active support elements. The critical deformation of rock mass prior to caving is much higher than the ultimate load bearing capacity of cables, therefore, a pre-tensioned cable element might not serve any useful purpose.

b) Several theories based on the formation of a compression zone (or arch) in the immediate roof above ceiling due to tensioned bolts, have been developed. This zone acts as a structural beam capable of taking load i.e. the proper reinforcing function

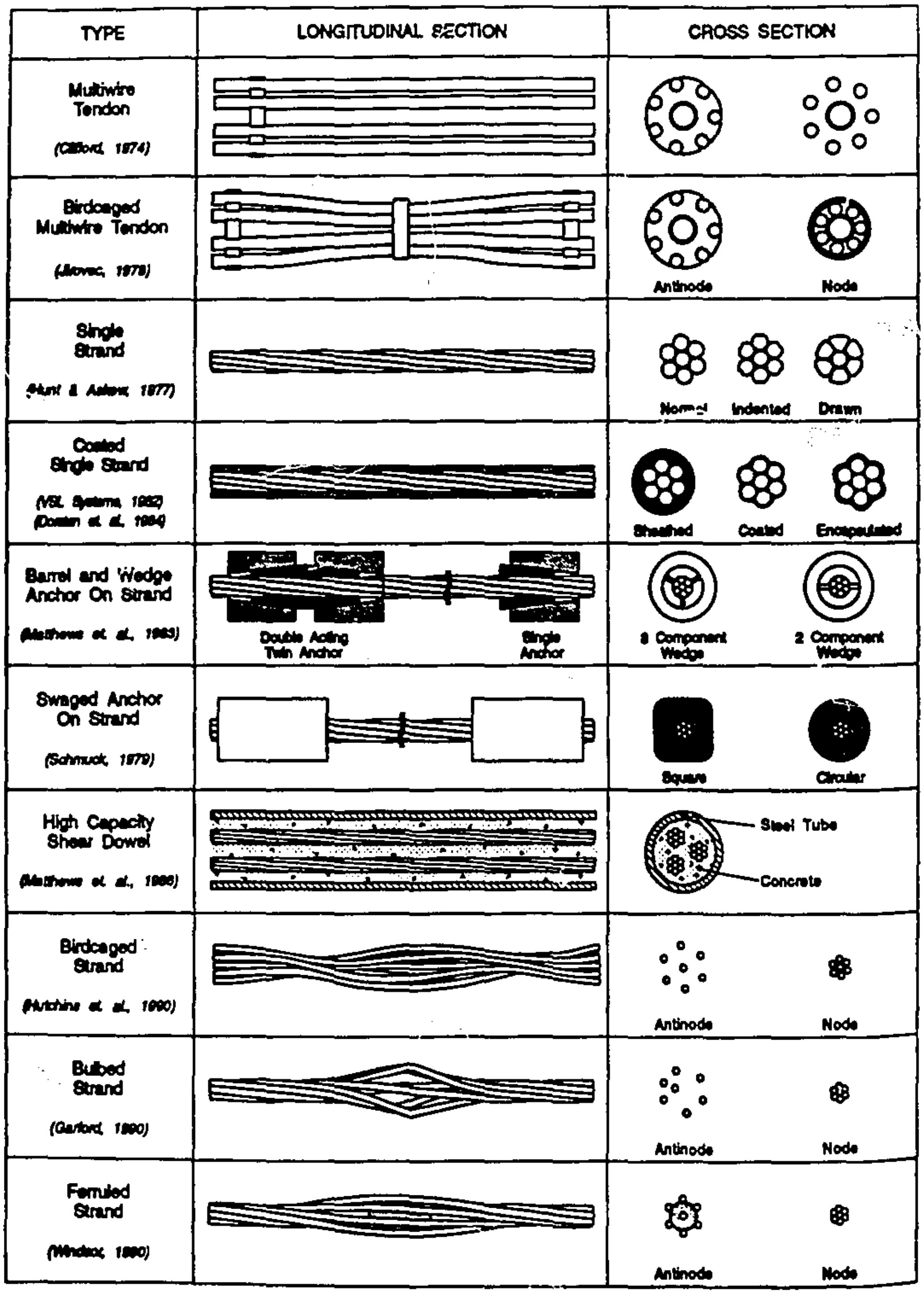


Figure 2.12: Summary of the development of cable bolt configuration (Windsor, 1992)

bolts. These concepts were advocated by Panek (1956) and Lang (1962) and at one point these authors were at a turning point of the development of rock reinforcement theory. This theory, however, is not accepted today. It is now a well established fact that the rockmass above the excavation forms a natural horizontal confined arch (Bergman and Bjurstrom, 1983). This arch, as a matter of fact, work on the principal of a self-supporting and controlling of dilatency of joints. Thus a point load, which is to support the arch from inside the opening, in fact, it is the most devastating load that can disturb the complete equilibrium. Thus, long tensioned cables, reaching for above the natural arch do not act as a prospective support to control dilatency in fractured rock mass. Moreover, the compression arch created by tensioned bolts do not give any practical advantages, but on the contrary, disorders the ideal natural pattern.

2.2.4 Mining Applications of Cable Bolts

Cable bolting is primarily used as temporary support in cut-and-fill stopes, open stopes and in stabilizing crown pillars. Combination of cable bolting and steel wire mesh is an effective tool against rock bursting prone areas. More than these conventional applications, cable bolting is also applied in hardrock mining to reinforce rock mass in close vicinity of opening. Techniques such as using two cables in one-cable-drill-hole with end plate, etc. are also gaining popularity. The following is the brief description of principal applications of cable-bolting in underground mining:

Cut-and-Fill Stope

Cable bolting in cut-and-fill stope is used to reinforce the backs of the stopes. The main function of fully grouted cable is to reinforce the rock mass immediately after the excavation of the stope before the ground gets relaxed. The formation of failure started with the inward movement of hanging wall. This leads to the dilation of cracks behind the immediate layer of the stope wall (Fuller, 1981). This means that cable element should be stiff enough to control the relaxing of ground.

Pre-reinforcement of stope eliminates the need for pre-tensioning of cables, provided enough compressive stress exists in the back. In regions where horizontal stress is low, however, pre-tensioning may be essential to provide additional confinement to the rockmass. Combination of cables and tensioned bolts can be an alternative. In such a case, tensioned rockbolts will create an artificial rock beam and cable bolts will suspend this beam with intact rocks. Application of cables in cut-and-fill stope is shown in Figure 2.13.

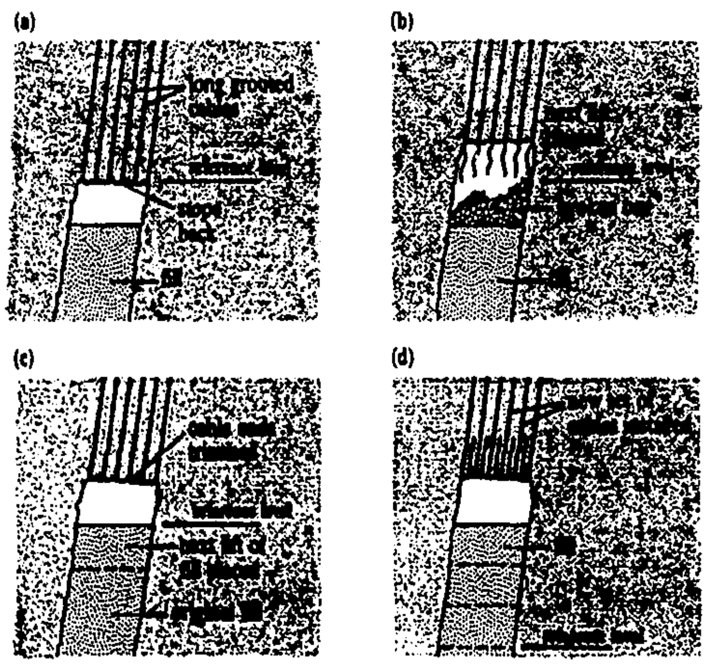


Figure 2.13: Application of cables in the stage by stage development of cut and fill stope (Brady and Brown, 1985).

Open Stope

In open stopes, cable bolting is mostly used to reinforce the hanging wall. The main purpose is to make open stoping methods possible even in poor rock/stress environment and/or decrease ore dilution (Davidge et al., 1988). The function of the cable bolt reinforcing systems in cut-and-fill and open stope methods is different. In open stoping, the exposed ground areas are larger and minor failures can be tolerated with smaller and irregular support densities (Fuller, 1983). Moreover, the stress levels and the rate of loading are greater in the case of long hole open stopes (Matthews et al. 1983).

Fuller (1983), after describing the principles for cable design in open stope walls, recommends that in low stress environment the cable bolt stiffness should be maximized, but in highly stressed areas the rockmass has to be allowed to yield progressively. It seems that mainly because of the large exposed areas, deformations at the hanging wall are large and therefore the main characteristic of cable bolt systems is to have high residual resistance. Such flexibility must be suitable to high deformations of ground. In other words the cable bolt element has to exhibit yielding (or slipping) characteristics compatible to the local ground conditions. Such an example is shown in Figure 2.14.

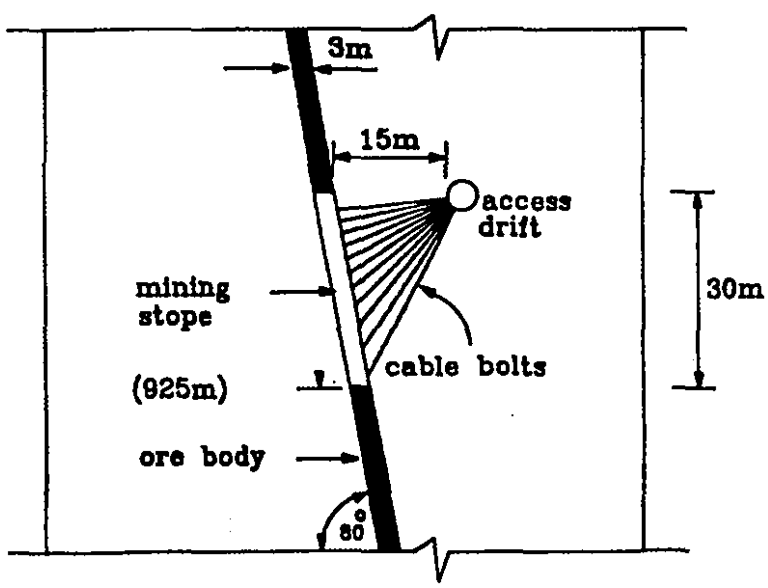


Figure 2.14: Stabilizing of hanging wall of long flexible cables (Mitri & Rajaie, 1990)

Pillar Support

Due to high stress concentration induced by mining, the potential failure of a pillar can appear in the shape of cracks and crushing. The post failure resistance in conjunction with the consequential regional stress relaxation gives stable conditions. For this reason, reinforcing a pillar with cable, in fact, does not prevent its failure but prevents the desecration of the already failed rock. Consequently, the main characteristic that the cable support element must exhibit is the ability to yield and control the deformation. Cables are fully grouted or wound around the pillars, as shown in Figure 2.15.

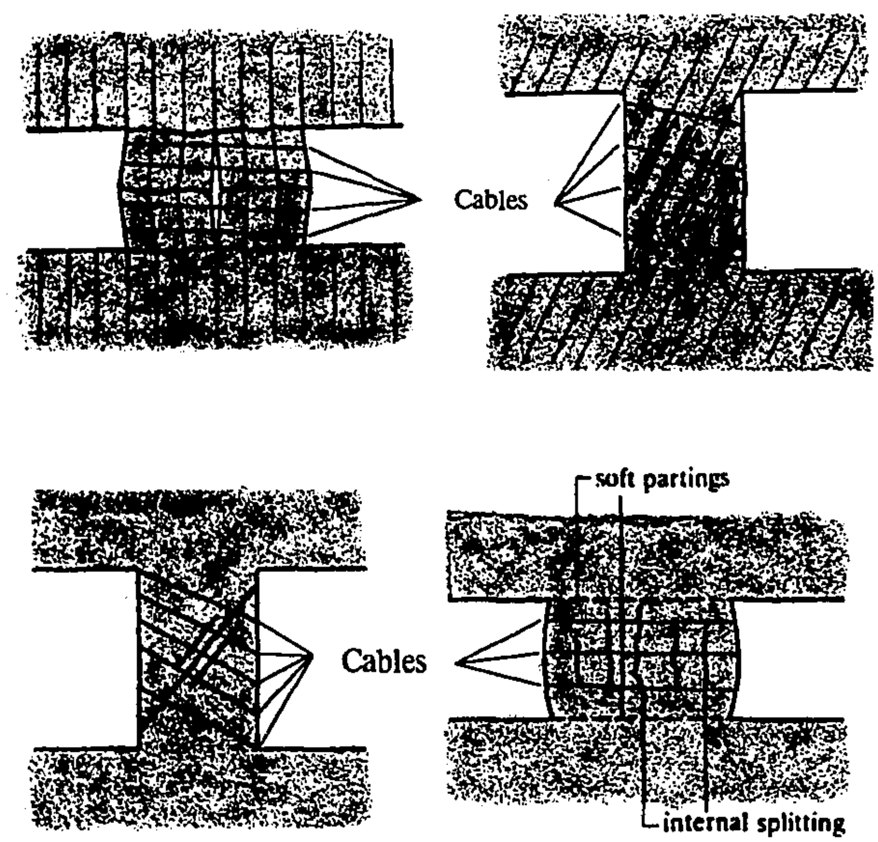


Figure 2.15: The use of cable bolts in pillar reinforcement (modified after Brady & Brown, 1985)

Rockburst Control

The additional requirement that a support system has to exhibit in a rockburst prone environment is the ability to yield at high strain rates (Davidge et al., 1988), for example see Figure 2.16. The use of cables currently employed in the form of lacing is mainly flexible external support system that aims to reduce the consequences of after-effects from a rockburst event.

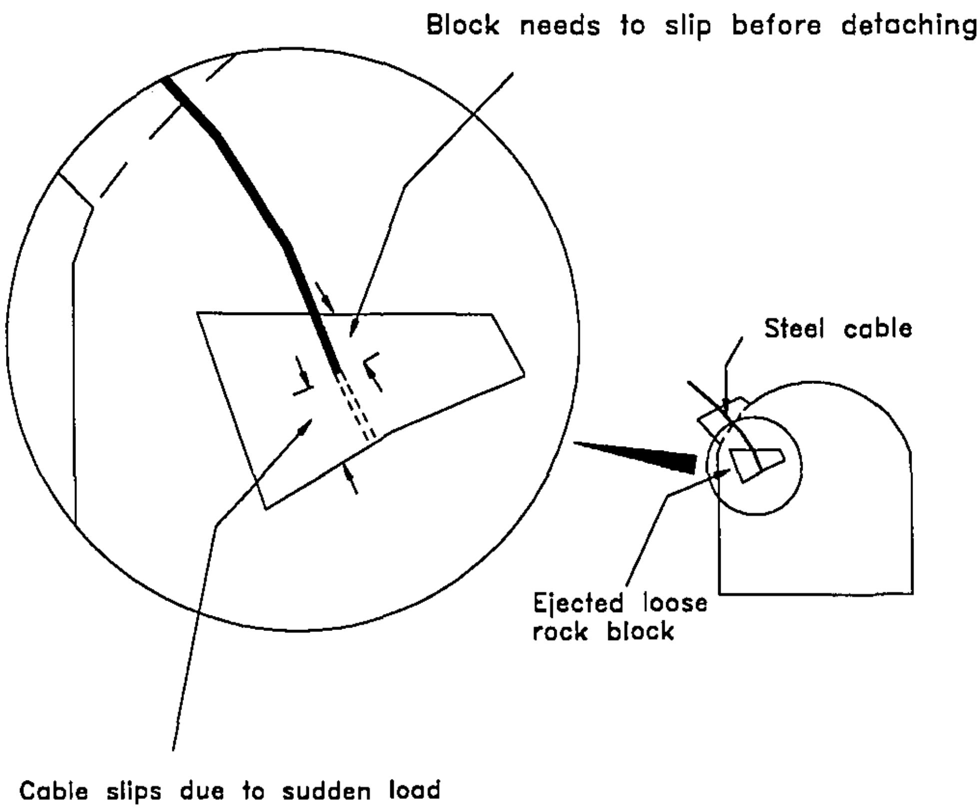


Figure 2.16: The concept of a dynamic shock absorbing capability of a cable during a rockburst

2.3 Mechanical Behaviour of Fully Grouted Tendons

2.3.1 Definitions

The behaviour of fully grouted tendons in mining applications are synonymous with the rebars and prestressing strands in civil engineering. The basic difference in these two fields is that in mining, boundary conditions being the rockmass, are considered as infinity, whereas in civil engineering, the clear cover or the maximum concrete around the rebar is taken as its boundary. The predominant function of tendons in mining is to contain uniaxial tensile stresses while in civil engineering applications, rebars and strands are mostly used in flexural and tensile modes. Tendons can be divided into the following three categories:

1. Smooth rebars. This type of tendons has a smooth surface, therefore, the bonding between bar and concrete primarily depends on chemical addition and friction only. The behaviour of smooth rebars is completely different from smooth flexible cable (strand), and therefore, these can seldom be generalized in one category.
2. Deformed rebars. Their mechanical behaviour, basically, depend on the shape, size and orientations of aspirates/lugs. In general, most bond failures are due to the splitting of surrounding matrices. During the process of pull out, load from the lugs/aspirates of the rebars is applied as a combination of tensile and shear on the matrix, which propagates cracks and eventually split the grout around the tendon. The load carrying capacity of cracked or splitted grout is zero or very small, therefore, the load carrying capacity of such rebars is very minimal in the post-elastic region.
3. Flexible cables. The 7-wire strand or cable bolt has a unique cable-grout interaction behaviour. The bonding between cable and grout depends upon chemical adhesion, frictional resistance and mechanical interlocking. The chemical bonding is of secondary

nature, while friction and mechanical interlocking are responsible for its extended load-carrying behaviour. In the elastic limit, the behaviour of deformed rebar and cable bolt is more or less same, however, the load sustaining capacity of cable bolt in the post-elastic region is completely different from its counterparts.

2.3.2 Distribution of Shear Bond Stress

Hawkes and Evans (1951) performed pull out tests on steel bars (referred as mild steel, in their paper), which were used then in concrete columns and beams. They concluded, from their pull out experiments, that the distribution of bond stress along the bar is an exponential function and that the maximum value of bond stress develops at the loaded end of steel before any slip takes place. After initial slip, the position of maximum bond stress moves along the steel. Furthermore, they found that stress in steel is proportional to the bar-concrete shearing stress before debonding/slip starts. This theory, however, exclusively developed for rebars and can not be generalize for steel cables. Along with Figure 2.17, the following are the mathematical expressions of bond distribution were put forward by the authors:

$$\tau_x = \tau_o \, e^{(-2\frac{Ax}{r_c})} \qquad 2.6$$

$$P = \frac{\pi r_c^2 \tau_o}{A}\{1 - e^{(-2AL/r_c)}\} \qquad 2.7$$

Where:

x = distance along the bar from the loaded end;

A = slope of the variation of normal stress of anchor with respect to the bond stress;

τ_o = shearing stress at $x = 0$;

τ_X = shearing stress at x distance from the surface;

P = applied load at one end of bar (pulling load);

r_C = radius of the cable bolt;

L = length of specimen.

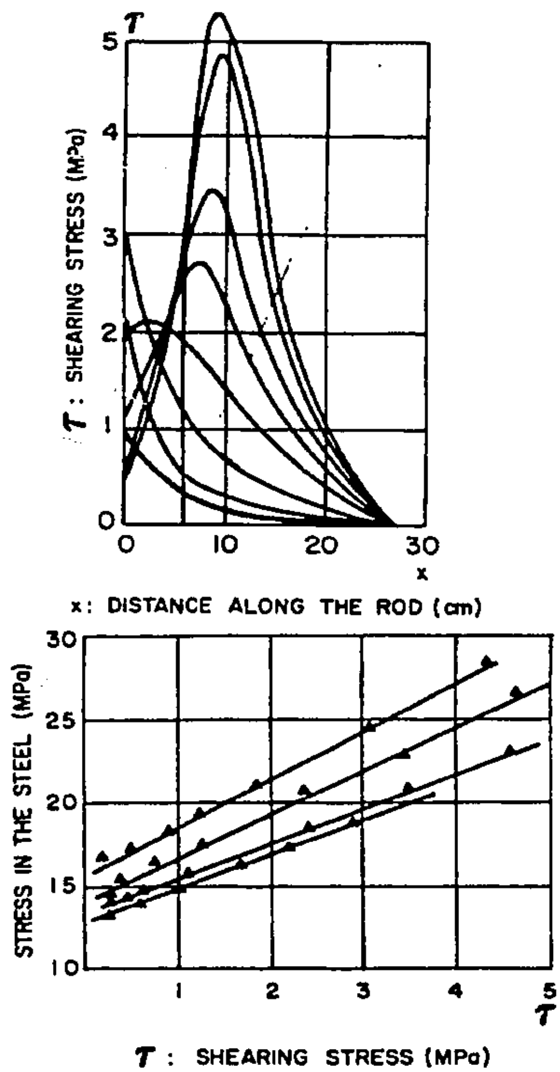


Figure 2.17: Shear stress distribution along the embedment length of fully grouted rebar (Hawkes and Evans, 1951)

Coates and Yu (1970) took the above equations and proved that Hawkes and Evans (1952) approach was applicable to the anchorage in rocks. In their approach, Coates and Yu used finite element method to approximately calculate the stresses around an anchor in a cylindrical hole in a triaxial stress field. The system was modelled by two sets of materials with different elastic moduli. The analyses were limited to elastic and no attempt was made to see the response in the post-elastic zone. They concluded that for soft rocks, a uniform distribution of stress along the embedded length of bar exists, whilst, for medium hard and hard rocks, the exponential distribution of stress was obtained. The soft rocks were defined by the authors as: Ea/Er > 10 and USC < 7 MPa.

Where:

Ea = elastic modulus of tendons (bar);

Er = elastic modulus of rock;

UCS = uniaxial compressive strength.

At the same time, Littlejohn (1970) after conducting pull out test on different types of soils, produced a table of empirical relationship between pull out force 'P' and parameters such as anchor dimensions (length L, diameter d), relevant soil properties (cohesion C, angle of internal friction φ, etc.) and depth of overburden. Littlejohn's experiments, however, were limited to soil mechanics only.

Philipps (1970) simplified the theory of Hawkes and Evans by defining the constant 'A' of equation 2.1 and 2.2. He suggested that for A < 0.01, the stress distribution is approximately uniform, and for A > 0.01 is exponential, as shown in Figure 2.18. Phillips also hypothesized a linear relationship between Ea/Er and 1/A. Most of his work was on hard rocks. With Phillips theory, equations 2.1 and 2.2 become:

$$\tau_x = A\frac{P}{4\pi r_c^2} e^{(-Ax/2r_c)} \qquad 2.8$$

$$P = \frac{4\pi r_c^2 \tau_o}{A}\{1 - e^{(-Ax/2r_c)}\} \qquad 2.9$$

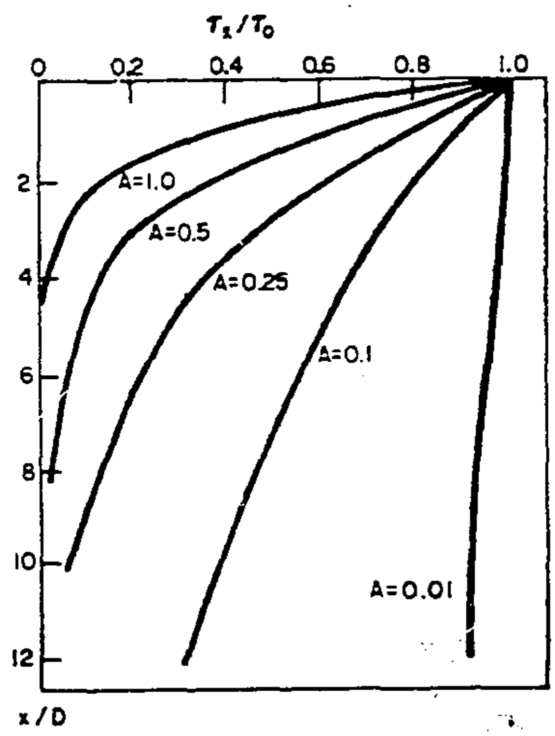


Figure 2.18: Stress distribution along the embedment length and affect of parameter 'A' (Coates and Yu, 1970)

By making use of pull out tests conducted by Ontario Hydro (Brown, 1970), Hollingshead (1971) modelled three phase material systems (steel tendon, grout and host rock) by finite element technique. He used an elastic-perfectly-plastic analysis assuming each of the three materials to yield according to the Tresca yield criteria. In his axisymmetric finite element modelling, he obtained yield zone with increasing load. The yield zone started from the loaded end and no element of tendon or host medium reached to the yield zone. Within the limitations of assumed materials, he observed that the bar could not be pulled out until the last element yielded, and by this time, the shear stress was uniform along the bar and equal to one-half the yield stress. His results are shown in Figure 2.19. From this figure, it is seen that shear stress distribution at grout-bolt interface along the embedded length is exponential in the elastic limit. When the shearing stress exceeds the shearing strength of interface, a debonding started occurring between bar and grout and, as a result, shear stress moves deeper along the length.

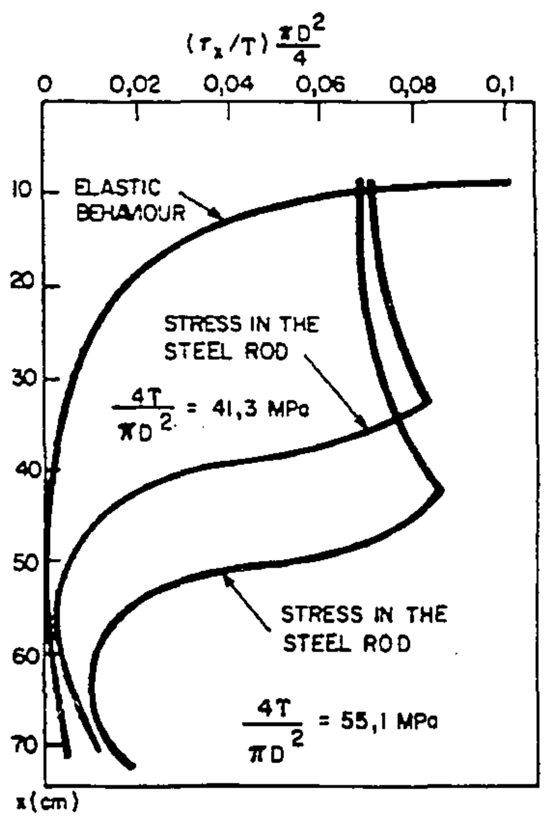


Figure 2.19: Calculation of distribution of stresses along a grouted tendon using finite element technique (Hollingshead, 1971)

Farmer (1975) has conducted pull out tests of rock anchors using concrete, limestone and chalk as host medium. Organic grout i.e. polyester resin was used to fully grout the tendons in pre-drilled holes. Farmer developed an analytical model (see Figure 2.20) by assuming host rock as rigid. He concluded that if boundary conditions were satisfied, then the shear distribution along the embedded length was non-linear for hard rock prior to tendon yielding. The shear distribution τ_x along the length was given by the following equations:

$$\tau_x = \frac{\alpha P}{2\pi r_c} e^{(-\alpha x)} \qquad 2.10$$

where:

$\alpha^2 = \dfrac{E_g}{E_c(r_g - r_c)}$; and subscript 'c' stands for tendons and 'g' for (outer diameter of) grout.

Farmer modified his equation for resin grout and came up with the following equation (see Figure 2.21):

$$\tau_x = 0.1\ \sigma_o\ e^{-(2x/r_c)} \qquad 2.11$$

where:

σ_o = applied axial stress at load end face.

Farmer further confirmed the existence of exponential distribution relationship for an Ea/Er = 0.1 and 0.6 and uniform distribution for Ea/Er = 0.75. In short, his work showed that failure of grout-tendon interface was accompanied to the loss of cohesion or shearing of the interface.

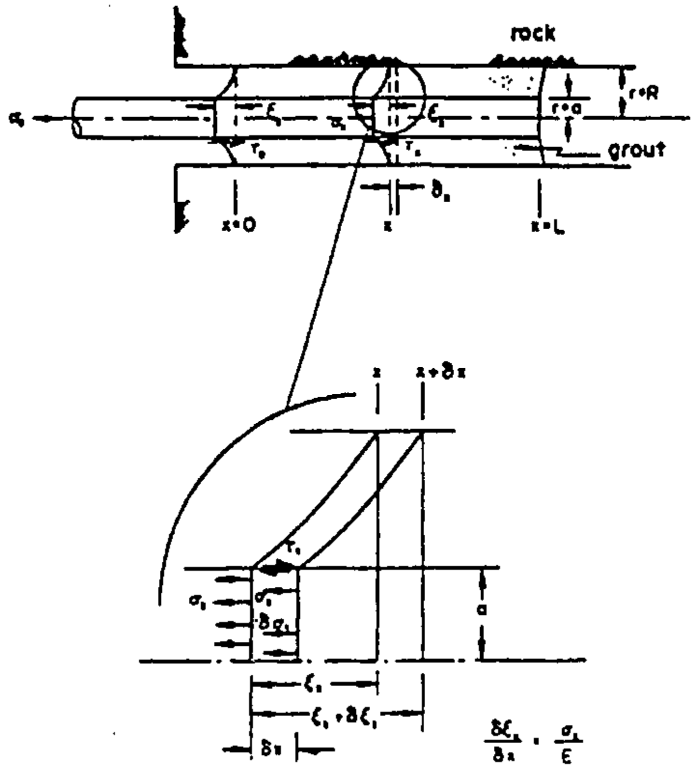


Figure 2.20: Distribution of stresses along the fully grouted tendons (Farmer, 1975)

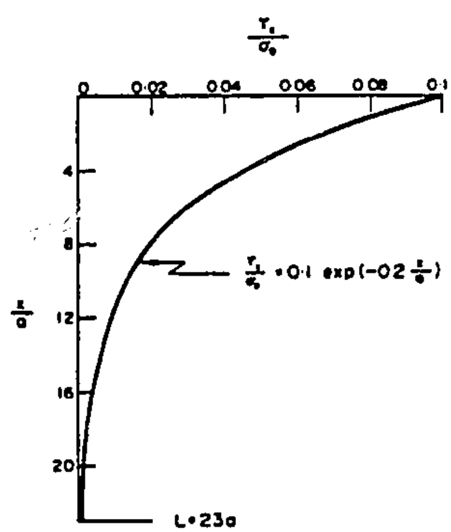


Figure 2.21: Theoretical stress distribution along a fully grouted (resin) anchor in rigid hole with a thin grout annulus (Farmer, 1975)

Dunham (1976) used theoretical formulation of Farmer and compared with his experimental results. Pull out tests were carried out using 25 mm diameter ribbed steel bars (resin) grouted in 36 mm diameter holes. Two different embedment lengths 300 and 750 mm were used in the experiments. From the rest results, Dunham concluded that "the shape of shear stress distribution curves remain sensibly exponential however, probably due to small movements which have occurred being insufficient to overcome the high level of skin friction". His further conclusions were that at low applied loads, the initial stress distribution was close to the theoretical results. Whereas at higher loads, debonding of the anchor has occurred over at least half the anchor length and the shear stress was disturbed according to the relative movement. Therefore, the degree of frictional shear strength of interface is mobilized.

Benmokrane and Ballivy (1982) studied the mechanism of anchorage failure and the shear stress distribution along the fully grouted rockbolts. In their research, they included parameters such as grout-bolt interface bonding, application of constant load at bolt head (i.e. creep test) and the behaviour of bolt under various loading conditions. They suggested the following equations to find out the shear stress distribution along the tendon-grout interface:

$$\tau_x = E \frac{d}{4} \frac{d\varepsilon_x}{d_x} \qquad 2.12$$

where:

ε_x = axial strain in the tendon

They instrumented seven fully cement grouted rock anchors (bolts) in the field and measured the strain variations during pull out tests. According to them, shear stress distribution depends on the slope of the strain curves along the tendon. Figure 2.22 shows the distribution of axial strain in the bolt obtained during the pull out testing.

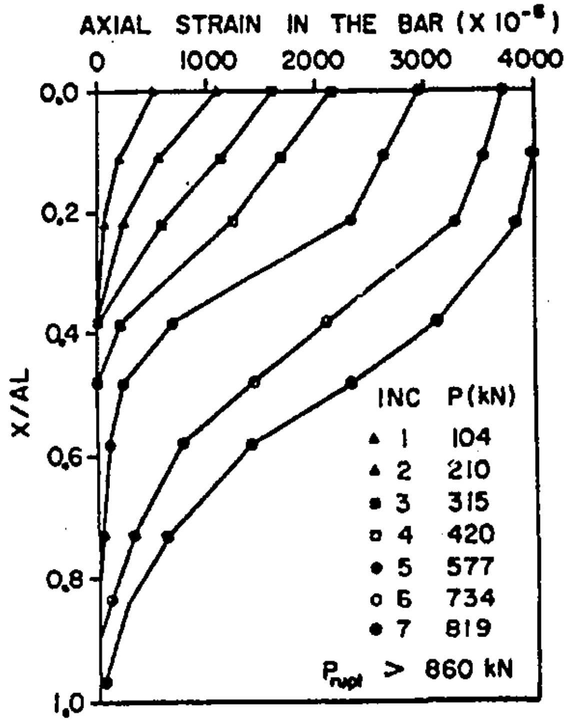


Figure 2.22: Distribution of axial strain in the bolt during pull out testing (Benmokrane & Ballivy, 1982)

John and Van Dillen (1983) developed a new one-dimensional element allowing yielding only at the bolt-grout interface. This one-dimensional element can be incorporated as a rock bolt in any general purpose, non-linear finite element computer code suitable for rock modelling. The limitation of this element is that it does not consider any radial stress around the borehole and, no consideration was given to the diletancy of the interface and contractancy of the steel bars.

Ballivy and Martin (1983), based on the work of Hawkes and Evans, Phillips and Hollingshead, formulated a model of stress distribution for the steel-grout and rock-grout interfaces; as shown in Figure 2.23. The maximum permissible load for the anchorage, as seen from the figure, can be determined when the surface below the τ versus x curves is maximum. They performed pull out tests both in laboratory and in field. In laboratory, they used conventional deformed steel rod, 9.5 mm diameter, with varying embedment length from 43 mm to 120 mm. Host medium was modelled by concrete and granite blocks. In fields, a 3.58 cm deformed steel # 11 rod was fully grouted in granitic gneiss rock. The interpretation of result was that rod-grout failure was due to shearing failure of the grout or to a compressive failure of the grout links at 45^{o} (see Figure 2.24). The anchor failure was based on the elastic limit of rod. They explained the phenomenon in the following words: "In order that the grout to be formed by a system of links at 45^{o} working in uniaxial compression, it is necessary for the rod to have a radial contraction. If the rod is only slightly distorted, then the links of the grout are subjected to a triaxial system of stress. In theory, then it is possible to get an τ_{max} adhesion stress, the value of which is greater than $\sigma_{c/2}$, if a steel rod with a very high elastic limit is used and if the breaking of the first link is prevented, thus confining the following limit. The loss of linearity in the load versus distortion curve corresponds to the elastic limit of the steel and the breaking load corresponds to the grout break."

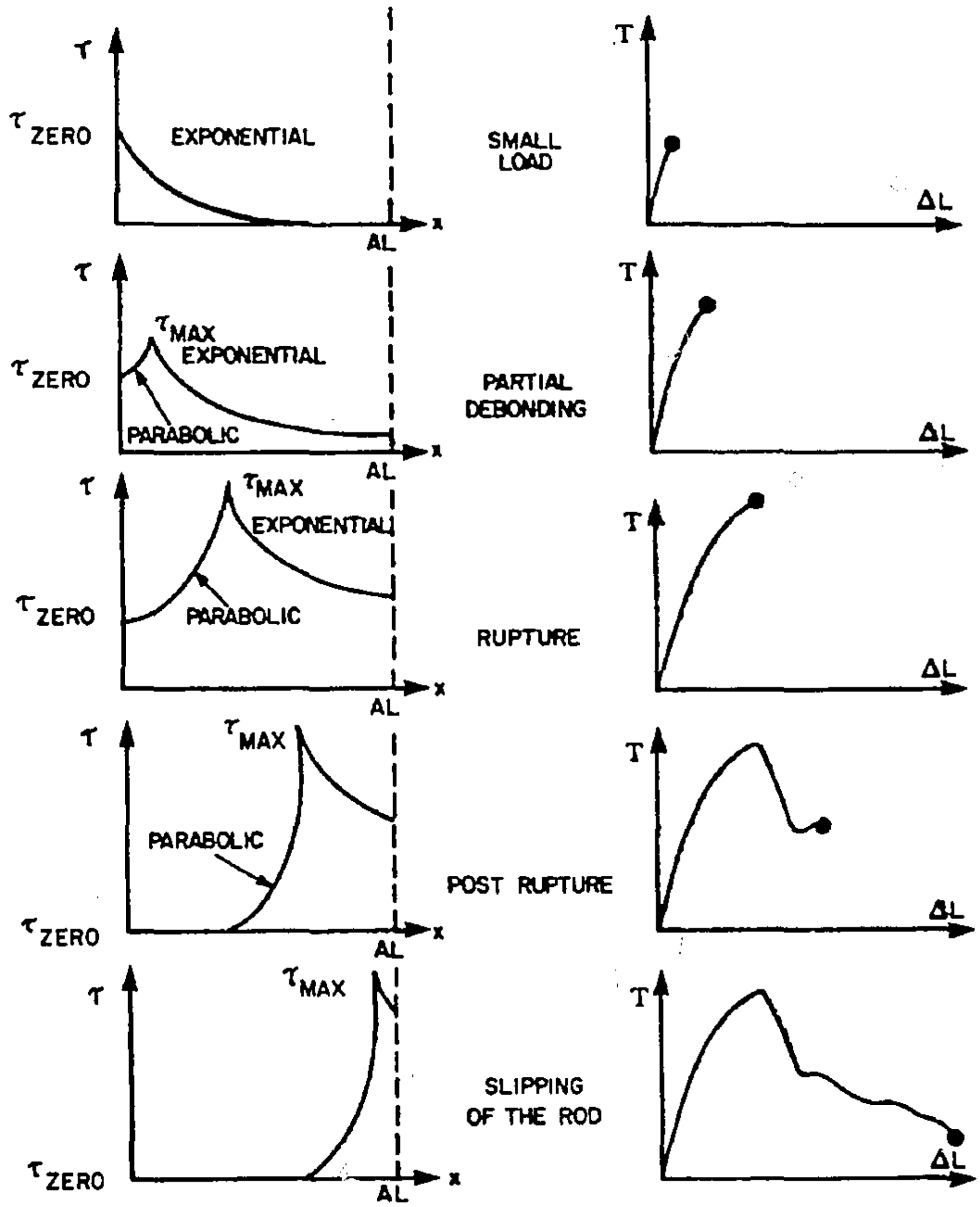


Figure 2.23: Gradual increase of load and corresponding shear stress distribution along the length of fully grouted tendon (Ballivy and Martin, 1983)

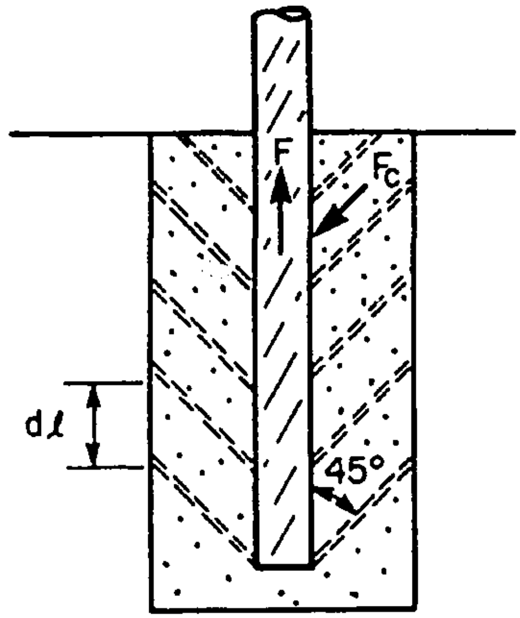


Figure 2.24: Compression-link-model of grouted portion of rockbolt system (Ballivy and Martin, 1983)

Fuller (1983) used cement based grouts to conduct pull out tests on untensioned seven wire conventional cable. A polythene or nylon air bleed tube was used to take out entrapped air at the space between cable and borehole. Fuller summarized his results as follows:

- the steel-cement bond (adhesion) failed at a low load and small displacement between the wire and grout;
- where bond failure has occurred, friction between the failed surfaces contributed to the pull out resistance;
- this friction contribution was very sensitive to even small variations in diameter of the wire along its length;
- a layer of surface rust and surface indentations in the wire substantially increased the bond strength.

Yap and Rodger (1984) used finite element technique to evaluate the mechanism of the transfer of load in the pull out tests of fully grouted tendons. They used three phase material systems to model bolt, grout and host rock. In formulating the FE model, they assumed the stress-strain behaviour of interfaces as elastic-perfectly plastic. In their isoparametric element formulation, however, they employed different yield criteria depending upon the type of materials. In the FE analysis, non-proportional loading (actual load increments) was used. During each load increment, each element in the mesh was checked for yielding with the under consideration yield criterion. If an element yielded, the excess stresses in the element were converted into forces to be redistributed. A maximum of 20 iterations were allowed for each load increment. The process of redistributing the load was stopped, and the next load increment was applied if either the deflection norm in a particular iteration was less than 10^{-4} times the total deflection norm, or if the load vector derived from excess stresses in the yield elements was less than 5 % of the applied force norm. Unlike other researcher, Yap and Roger concluded that the most severe position of shearing was at the rock/grout interface instead of bolt/grout interface and that the partial debonding was not a serious problem.

Aydan et al. (1985) presented a very comprehensive and detailed investigation of the behaviour of the development of shear stresses along the length of fully grouted rock bolt. A three phase element system with stresses and displacement around a rock bolt was used (see Figure 2.25). The behaviour of interface was assumed to be elastic-softening-plastic flow, as shown in Figure 2.26. In the analytical study, assuming that failure would take place at the bolt-grout interface, they presented the following formulation:

Shear stress τ_{rz} in grout:

$$\tau_{rz} = \frac{G_g}{r\ln(r_h/r_b)}(w_b - w_h) \qquad 2.13$$

with the following boundary conditions:

$$w_{rz} = -\,w_h \text{ at } r = r_h$$

$$w_{rz} = -w_b \text{ at } r = r_b$$

where:

G_g = shear modulus of grout
r_h = radius of drill hole
r_b = radius of bolt
w_{rz} = shear displacement
w_b = displacement of the bolt-grout interface and bolt in z-direction
w_h = displacement of grout-rock interface

The shear stress τ_{rz} in the rock was given as:

$$\tau_{rz} = \frac{G_r}{r\ln(r_o/r_b)}\, w_h \qquad 2.14$$

and the following were the boundary conditions for this equations:

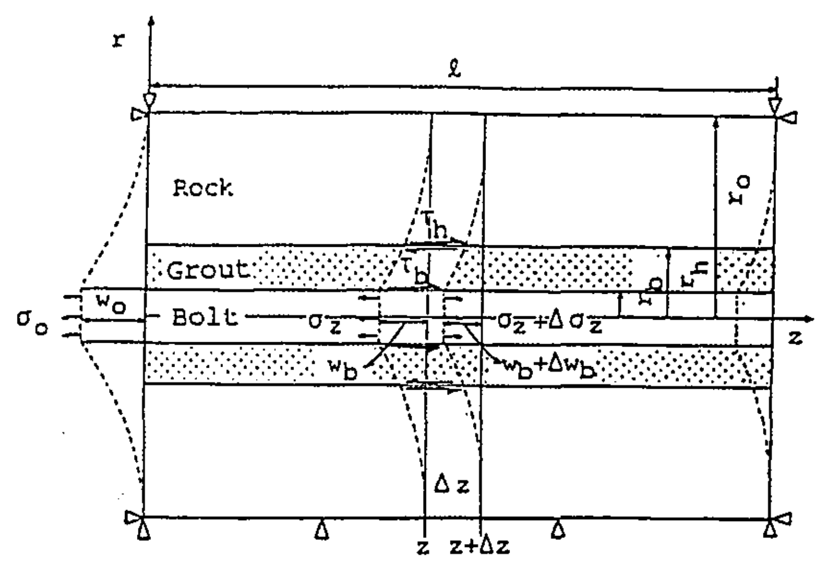


Figure 2.25: Stresses and displacements due to applied load in a fully grouted tendon (Aydan et al., 1985)

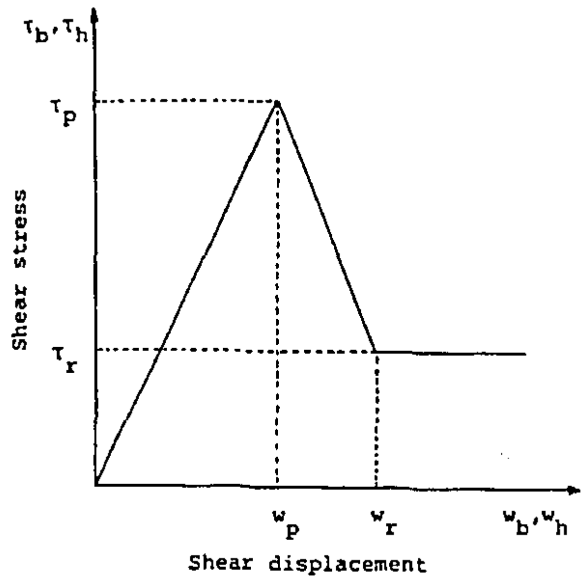


Figure 2.26: Shear behaviour of interfaces (Aydan et al., 1985)

$w_{rz} = 0 \quad$ at $r = r_o$

$w_{rz} = -w_h$ at $r = r_h$

where:

G_r = shear modulus of rock

r_o = radius of rigid boundary

The substitution of r for r_b and r_h in equation 2.13 and 2.14 yielded the expression for shear stress at the interface of bolt-grout, τ_b and shear stress at the grout-rock interface, τ_h as special cases of τ_{rz} respectively:

$$\tau_b = \frac{G_g}{r_b \ln(r_h/r_b)} (w_b - w_h) \qquad 2.15$$

$$\tau_h = \frac{G_g}{r_h \ln(r_o/r_h)} w_h \qquad 2.16$$

Aydan et al. also developed analytical relationships for residual (plastic flow) region, softening region and for elastic region (see Figure 2.26 for the definition of these regions). In the same paper, they described a one-dimensional coupled element. The features of this element are that it can simulate the pull out test of rock bolt and can also be incorporated in any general purpose finite element software. Authors argued that parameters such as effect of confining pressure of rockmass on the interface of fully grouted bolt could be taken into account as an increase in the peak and residual strength of the interfaces and a change in the values of the ratio γ of residual displacement to the peak displacement. Aydan et al. used their numerical and analytical formulations to analysis the parametric studies of rockbolts. They also compared their results with previous researchers and they made the following conclusions:

i. The physical properties of rockbolt, grout and rockmass greatly affect the stress distribution along the length of the fully grouted rockbolt. The effect of the elastic modulus of grout becomes insignificant once it exceeds the modulus of rock.
ii. A low shear stress is resulted if the hole radius is relatively increased than bolt radius.
iii. An elastic-perfectly-plastic behaviour of interfaces overestimates the load bearing capacity and reinforcement effect of rockbolt. A better result would be obtained with elastic-softening-plastic flow behaviour of interfaces.
iv. Yielding of interfaces started from the loaded end and progressed inward as the load increases.

Peng and Guo (1988) presented the formulation of fully grouted tendons by finite element technique. According to their theory, the total potential energy of a fully grouted bolt due to axial loads and displacements can be expressed by:

$$\varphi(u) = \frac{\pi D^2 E}{4} \{ \int_t [\frac{1}{2} (\frac{du}{dL})^2 - \frac{4}{DE} \tau u] dL - \frac{1}{E} \sigma_o u_o \} \qquad 2.17$$

where:

$\phi(u)$ = total potential energy of bolt;

u = axial displacement of the bolt;

D = bolt diameter;

E = weighted Young's modulus of the steel rod and grout material;

L = bolt length;

u_o = displacement at bolt head;

σ_o = normal stress due to any concentrated loading from either the bearing plate or pretension at the bolthead, and

τ = shear stress between the bolt and surrounding rocks.

The equilibrium condition of the bolt can be established by the variation of the total potential energy, i.e.

$$\delta\varphi(U) = 0 \qquad 2.18$$

After the bolt has been discretized into m finite elements, equation 2.18 can be written in the following algebraic form

$$G_a \tau = 1.5 \, D \, E \, K_a \, U + 1.5 \, D \, \sigma \qquad 2.19$$

where

$\tau = [\tau_o, \tau_1, \ldots, \tau_m]^T$, $U = (Uo, U_1, \ldots, U_m]^T$, $\sigma = \sigma_o, 0, \ldots, 0,0]^T$, and G_a and K_a are coefficient matrices of (m + 1) x (m + 1) defined by

$$G_e = \begin{bmatrix} 2l_1 & l_1 & & & & 0 \\ l_1 & 2(l_1+l_2) & l_2 & & & \\ & \ddots & \ddots & \ddots & & \\ & & & l_{m-1} & 2(l_{m-1}+l_m) & l_m \\ 0 & & & & l_m & 2l_m \end{bmatrix}$$

$$K_e = \begin{bmatrix} \frac{1}{l_1} & -\frac{1}{l_1} & & & & 0 \\ -\frac{1}{l_1} & (\frac{1}{l_1}+\frac{1}{l_2}) & -\frac{1}{l_2} & & & \\ & \ddots & \ddots & \ddots & & \\ & & & -\frac{1}{l_{m-1}} & (\frac{1}{l_{m-1}}+\frac{1}{l_m}) & -\frac{1}{l_m} \\ 0 & & & & -\frac{1}{l_m} & \frac{1}{l_m} \end{bmatrix}$$

where

l_i is length of the ith element.

Goris (1990), based on his experimental work of conventional cable, concluded that the average shear stress developed along the grout-cable interface could be determined for a specific load by dividing the load by contact area between the cable and the grout. He calculated the circumference of cable with the following equation:

$$C = N \times 3.14 \times D \times [(\sin(360/2N))/(\sin(360/2N) + 1)] \times (0.5 \times 1/N) \quad 2.20$$

Where:

C = circumference of the cable,

N = number of outer wire of the cable,

D = diameter of cable (D = 0.625 inch, (Stheeman, 1982).

For conventional cable, with N = 6 and D=0.625 inch (15.87 mm) the circumference 'C' comes out to be 2.62 inch (66.5 mm).

Using the concept of one-dimensional element of John and Van Dillen (1983), Itasca (1991) incorporated a rockbolt (or cable bolt) element in their finite difference software, known as Fast Lagrangian Analysis of Continua (FLAC). Axial behaviour of bolt and shear behaviour of grout annulus has been modelled as a system of rockbolt. The effect of shear resistance of grout along the length of fully grouted bolt can be obtained. The axial stiffness of bolt is described as function of cross-sectional area, A and Young's modulus, E. In formulation, a yield force is assigned to the cable, which checks and keeps cable element forces below this yield limit (for example, Figure 2.27). Moreover, in evaluating the axial forces development in the reinforcement, displacements are computed at nodal points along the axis of the reinforcement as shown in Figure 2.28. The shear behaviour of the grout is modelled as a spring slider system located at the nodal points (Figure 2.28). The shear behaviour of the grout annulus during relative displacement between the reinforcement/grout interface and the grout/rock interface is described numerically by the grout shear stiffness (Figure 2.28(b)). Numerical estimation for the shear stress, τ_G, comes from an equation describing the shear stress at the grout/rock interface (St. John and Van Dillen, 1983) :

$$\tau_G = \frac{G_g}{(D/2 + t)} \frac{(u_b - u_r)}{\ln(1 + 2t/D)} \qquad 2.21$$

where:

u_b = axial displacement of the reinforcing

u_r = axial displacement of the grout/rock interface,

G_g = grout shear modulus,

D = reinforcing diameter, and

t = annulus thickness.

The grout shear stiffness K_{bond} per unit problem thickness is simply given by

$$K_{bond} = \frac{2 \pi G_g}{\ln(1 + 2t/D)} \qquad 2.22$$

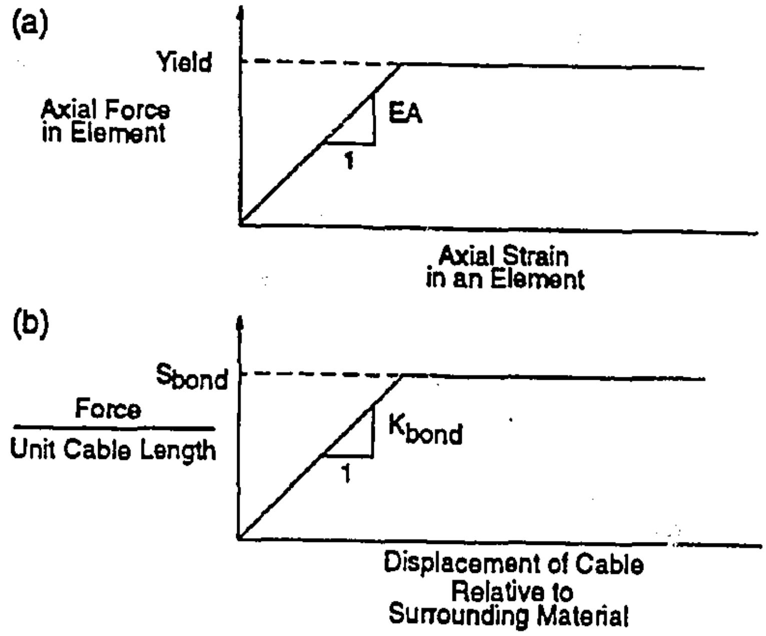


Figure 2.27: Axial (a) and shear (b) behaviour of cable elements (Itasca, 1991)

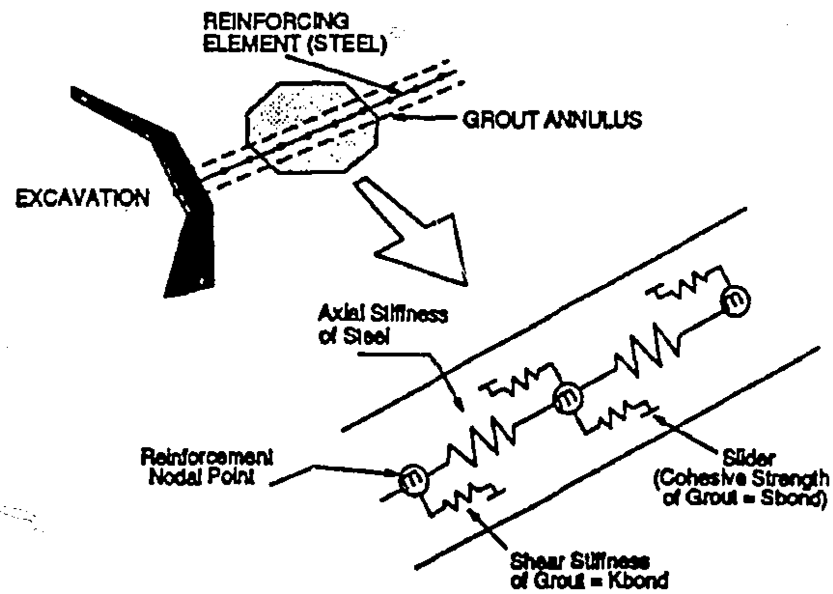


Figure 2.28: Conceptual mechanical representation of fully grouted reinforcement which accounts for shear behaviour of the grout annulus (Itasca, 1991)

The maximum shear force, that can develop at cable-grout interface, is limited by the product of the representative cable length and the parameter S_{bond} (i.e., maximum bond force/unit cable length).

Mitri and Rajaie (1990) developed a bolt element and incorporated into finite element model called MSAP2D (Microcomputer Static Analysis Program for 2-Dimensional problems). The formulation was based on the assumption that the grouting material could be idealized by a continuous, Winkler-type spring parallel to the fully grouted tendon and connecting the tendon surface to the borehole wall (see Figure 2.29). This spring model is capable of modelling the inherent slip at bolt/grout interface. This formulation, however, does not consider material and geometric non-linearities.

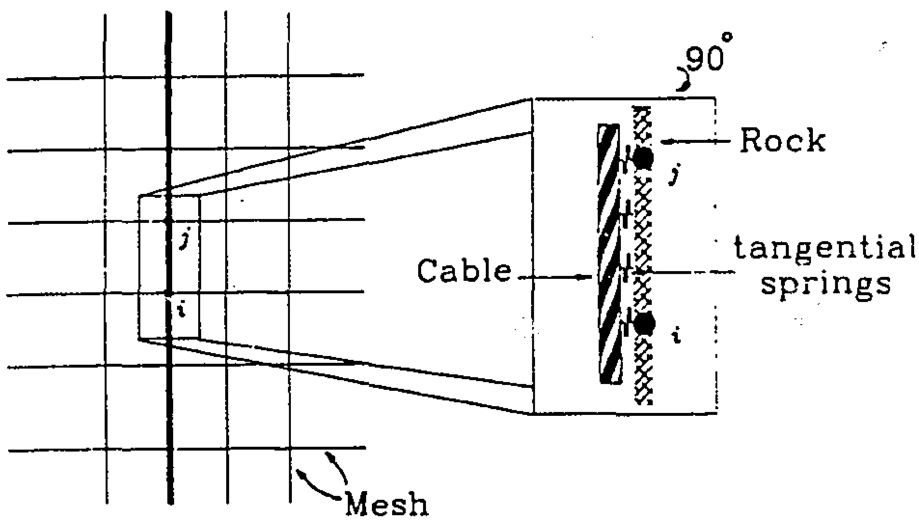


Figure 2.29: Finite element simulation of grout behaviour (Mitri and Rajaie, 1990)

UNWEDGE (Hoek, 1992) was another software which was basically developed for the analysis of structurally controlled wedge failure around underground opening. In addition to the option of selecting the potentially unstable wedges, UNWEDGE can also incorporate bolt elements. However, detail on the formulation of tendon element has yet not been published.

Wang and Garga (1992) developed a numerical model to include rockbolt element into a previously developed computer software BLOSMER (Block Model for Excavation in Rocks). Rock bolts were modelled as end-anchored which were simulated by a one dimensional deformable element or a spring (see Figure 2.30). In the formulation, two ends of the spring were assumed fixed on the rock blocks. With the stiffness of the bolt, K_b, the relative displacement between the two anchored blocks results in an axial force T in the bolt or mathematically:

$$T = K_b \Delta L \qquad 2.23$$

Where ΔL denotes the relative displacement between the two anchorage points, this could be assumed as the extension of the rockbolt. The bolting along the entire length is simplified as a series of equivalent point bondings (see Figure 2.31). However, the modelling of fully grouted tendon is oversimplified by this technique and analyses are limited to the elastic range only.

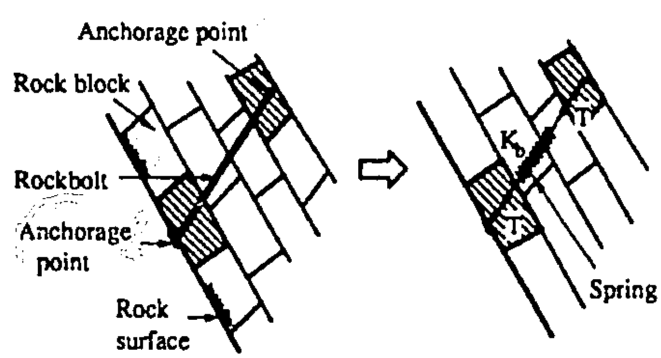


Figure 2.30: Simulation of end-anchored rock bolt (Wang and Garga, 1992)

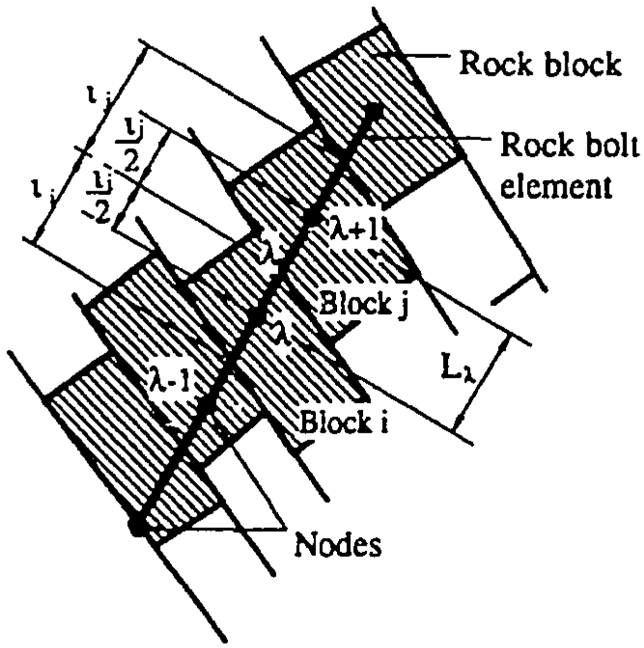


Figure 2.31: Modelling of fully-grouted rock bolts by bolt elements (Wang and Garga, 1992)

Yazici and Kaiser (1992) proposed a theory which explained the effect of deformation and strength properties of grout and rock, and geometry of the bolt and the borehole. With these parameters, they presented a theory for bond determination, combining the principles of dilative rock joints, pressure increase during elastic grout and rock expansion, and radial and axial failure of a brittle grout cylinder. A bond strength model which defined the behaviour of grout as (a) elastic, (b) fully split, or (c) partially split within elastic portion was presented. According to them, the radial displacement of grout during the axial (pull out) motion of cable, causes an internal pressure p_i which compresses the grout radially and eventually induces tensile tangential stress in grout (Figure 2.32). When the tangential stress reaches to tensile strength of the grout, a crack initiates at the bolt-grout interface. It propagates radially outward during further dilation and eventually may reach the grout-

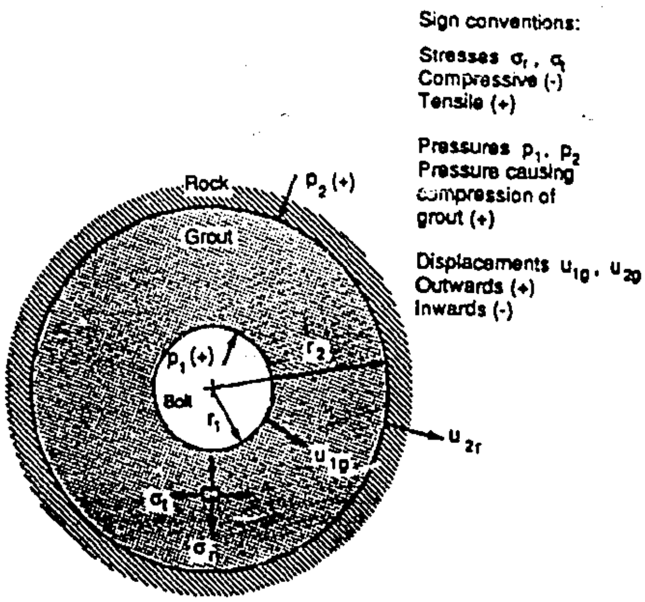


Figure 2.32: Conceptual cross-section through a fully grouted cable (Yazici and Kaiser, 1992)

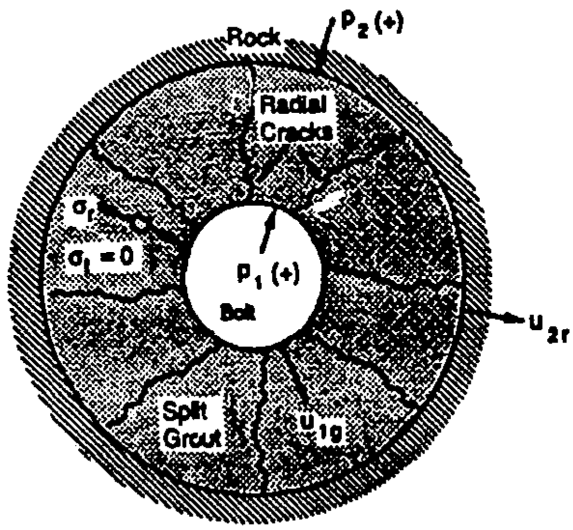


Figure 2.33: Conceptual cross-section through a fully-split grout column around a cable (Yazici and Kaiser, 1992)

rock interface. Once the grout splits, the tangential stress becomes zero and a new situation of crack propagation where the pressure p_i is fully transferred to the grout-rock interface as pressure p_i is created (Figure. 2.33).

Hyett at al. (1992) also suggested a failure mechanism of fully grouted conventional cable. They highlighted the importance of cement properties (i.e. grout), embedment length of cable and the radial confinement acting on the outer surface of the cement annulus. They varied different water cement ratios to see its effect on the pull-capacity of cable. Different types of pipes (i.e. steel, aluminum and PVC) were used to simulate the various stiffnesses of the host rockmass. In their failure mechanism analysis, they suggested that the load-displacement behaviour could be divided into definite stages, each associated with a specific failure mechanism as cable bolt failure proceeds. Their suggested theory is graphically represented in Figure 2.34.

Same mechanism of crack generation and grout failure has already been presented by Lucie (1992) but applicable for rebars only. According to her, the load carrying capacity of rebar, once the cracks have been generated, is very small i.e. close to zero. The failure pattern and propagation of cracks due to the asperities of rebar are shown in Figure 2.35

Malvar (1992) also presented the splitting of grout during the pull out process of reinforcing bars embedded in 76-mm diameter concrete cylinder. He applied various confining stresses around the concrete specimen and recorded its effect on the pull out behaviour. From Figure 2.36, it is shown that concrete has been splitted due to excessive cracking and consequently the load carrying capacity of grout diminishes.

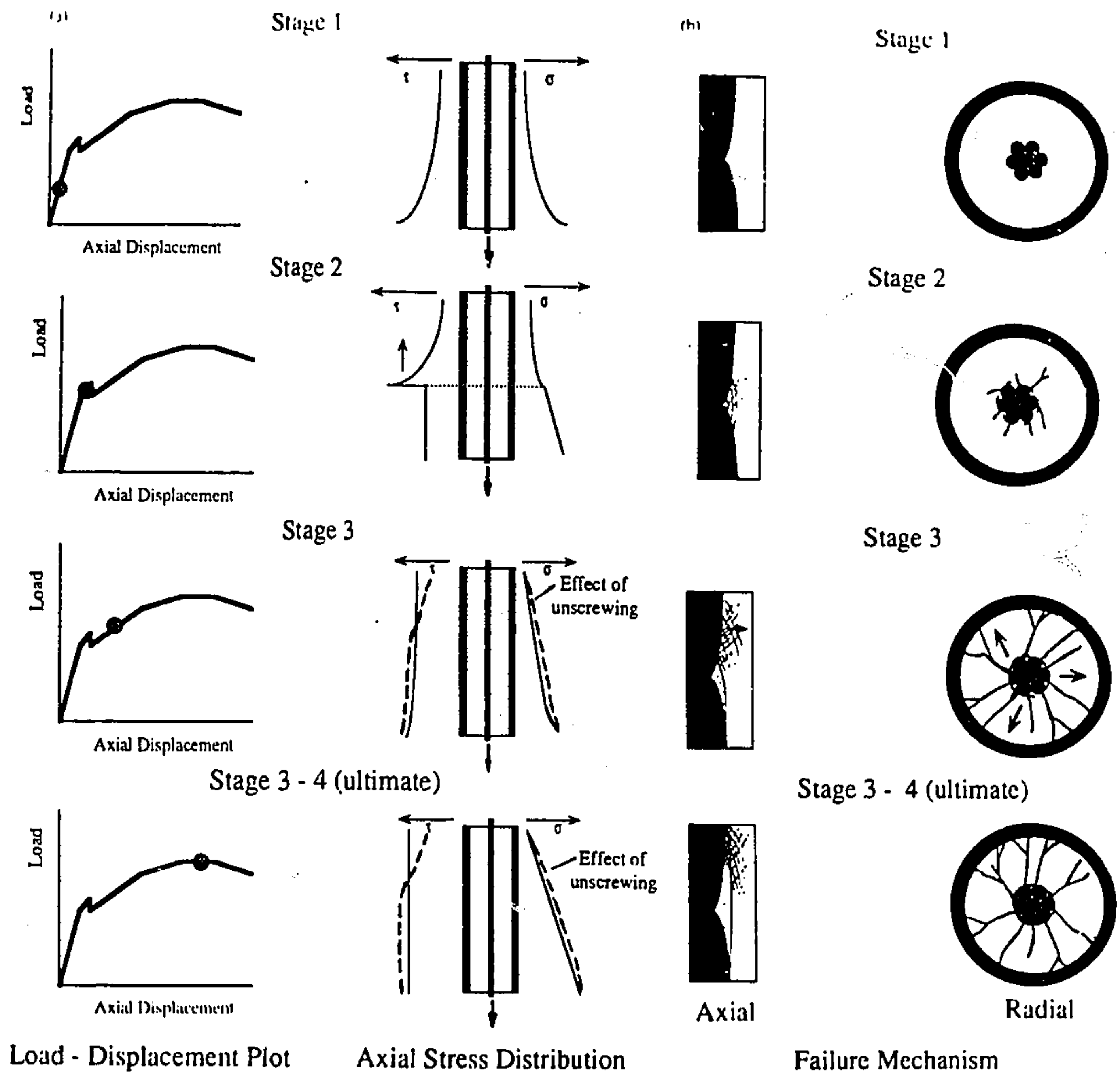


Figure 2.34: Pull out mechanism of fully grouted cable: (a) axial stress distribution in cable; and (b) crack propagation and failure mechanism of grout (Hyett et al., 1992)

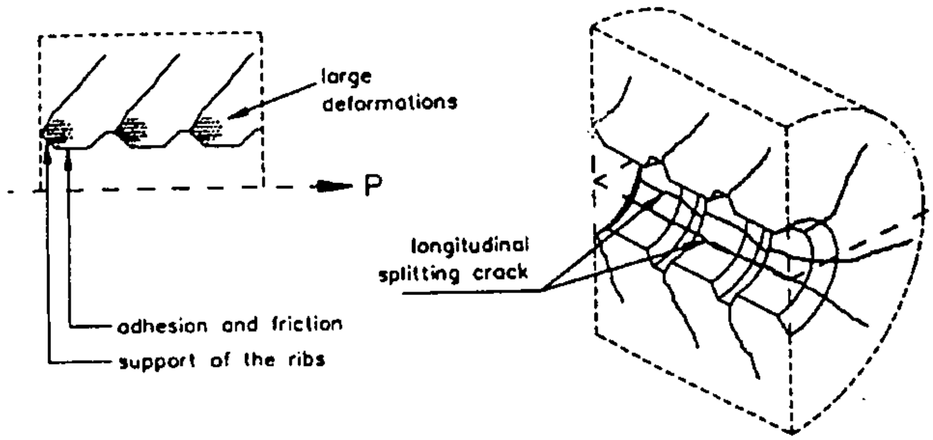


Figure 2.35: Pattern of cracks in the grout around a fully grouted rebar (Lucie, 1992)

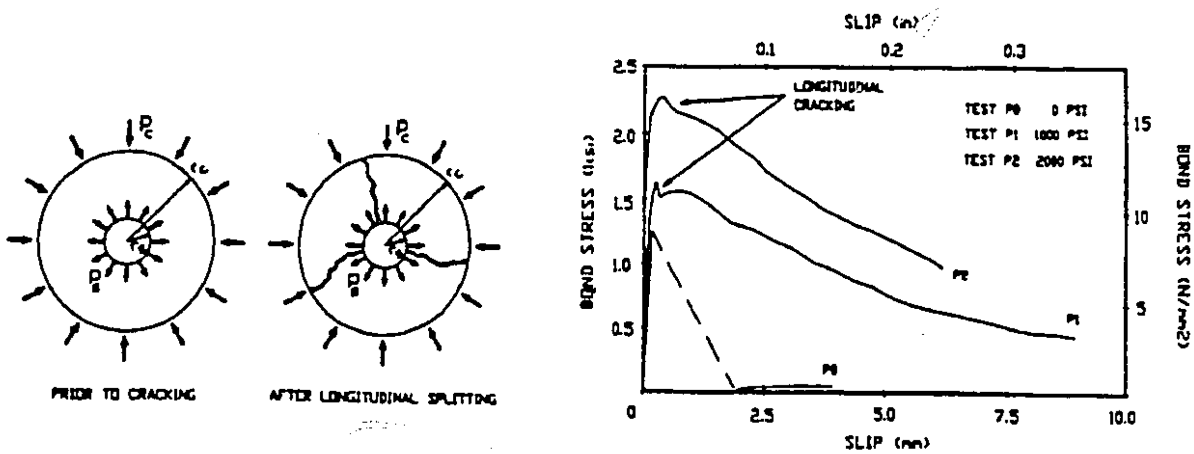


Figure 2.36: Confinement pressure and its effects on pattern of cracks and post-peak load carrying capacity of cable (Malvar, 1992)

Limitations of Analytical Models

The pull out behaviour of cable and bolt is similar in the elastic region, however behaviour as well as failure mode is completely different from each other in the post-elastic zone. A typical pull out behaviour of bolt and cable is shown in Figure 2.37. Considering the behaviour in view, the following are the main limitations of analytical and numerical models presented in the previous sections:

1. Most of the work is limited to the behaviour of fully grouted rebars. The mechanism of 7-wire strand or cable has not been exclusively analyzed and it is not differentiated from the rebar.
2. Except Aydan et al. (1985) and FLAC (Itasca, 1991), no provision for different behaviours of grout in the post-elastic range has been provided in the analytical and numerical models. The techniques of these two models can be extended to some extend to generalize cable pull behaviour in post elastic region.

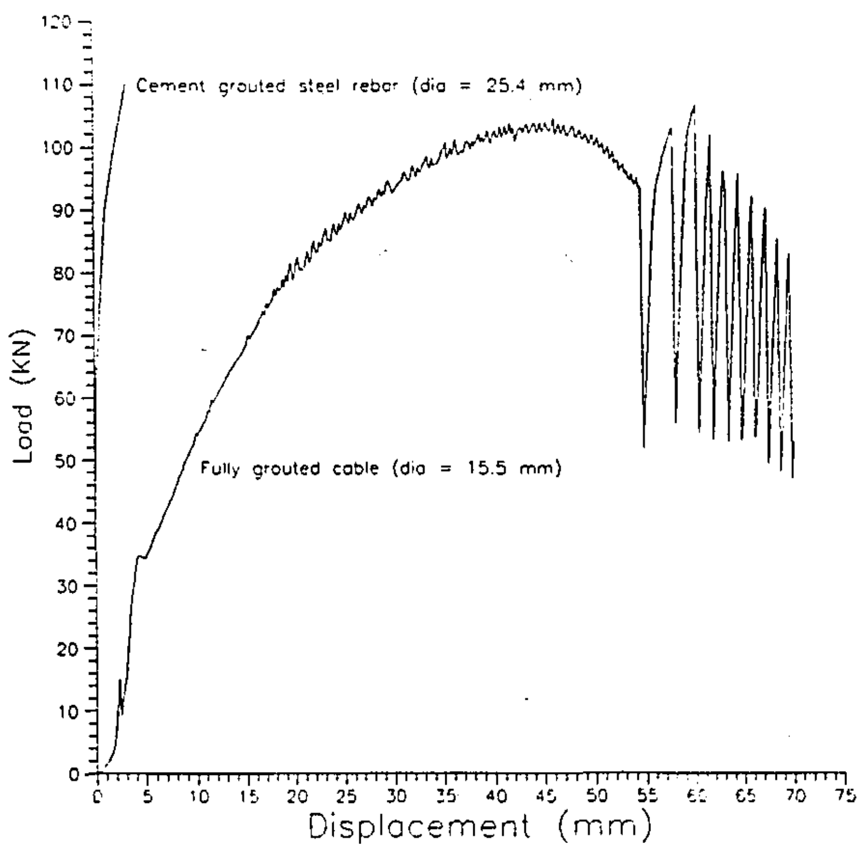


Figure 2.37: Typical pull out behaviour of fully grouted cable and bolt.

Chapter 3

Laboratory Investigation on Grouting Materials

3.1 Introduction

The overall performance of fully grouted cable bolt support systems depends largely on the quality of grouting (bonding) materials. Cement + water and organic grout such as polyester and epoxy resin have been used extensively as grouting agents. Cement grout is injected under pressure into the borehole, while, organic grouts which are produced in a sausage form, are inserted and then mixed in the borehole. There are a number of parameters affecting the mechanical behaviour of cement and resin grouts. These are discussed in the following sections.

3.2 Testing Program

In order to compare and evaluate the performance of fully grouted cables, the following grouting materials were included in the testing program:

- Conventional cement grout
- High strength cement grout
- Shotcrete cement grout
- Organic resin grout

Water-cement ratio proportions have been thoroughly investigated by various researches (Neville, 1981, Goris, 1990 and Rajaie, 1990). On the basis of their results, a 0.4 water cement ratio was selected, tested and found appropriate for both workability and strength.

Cardboard cylinder with internal surface greased and internal dimensions of 6 inch long and 3 inch in diameter, were used to prepare the grout samples. Samples were stored for 28 days in sample curing room at a temperature range of 20±4 °C with highly moist curing conditions. Samples were then removed from their moulds and end-conditions were fixed with automatic end-grinding machine. No sulfur end-casing was used, because it was not found compatible with high strength cement samples.

RDP-Howden electrohydraudiic servo-controlled testing system with SL 2000 analgue control consul, hydraulic power pack and Apple II micro-computer system was used for testing. A displacement control mode with a rate of loading of 1 mm/min was used for all uniaxial compressive tests.

The following mechanical properties of different types of grout were evaluated:

- Uniaxial compressive strength
- Modulus of elasticity
- Strain at failure

3.3 Grouting Materials

The following is a summary of the materials which were used in this study.

3.3.1 Cement

The main constituents of cement are silicates and aluminum of lime made from limestone and clay (or shale) which is ground, blended, fused in a kiln, and then crushed to a powder. Cement reacts chemically with water (hydration) to form a hardened mass.

Portland cement can have widely differing chemical compositions. The five standard types of Portland cement used in Canada are described in Table 3.1. Grout is a combination of cement and water while the same becomes concrete with the addition of fine and coarse aggregates. Concrete and grout, made with ordinary Portland cement, normally requires 14 days to attain adequate strength. However, the design strength of concrete or grout is reached after 28 days.

Table 3.1 Standard types of Portland Cement as specified by CSA [1977]

Type	Characteristics	Applications
10	Normal or ordinary cement	- General purpose
20	Lower C_3A (tricalcium aluminate) content	- Moderate sulfate resistance - Moderate heat of hydration
30	More finely ground and/or higher C_3S (tricalcium sulfate content	- High early strength - Low temperature concreting
40	Lower C_3S and C_3A content	- Low heat of hydration - Concreting massive sections
50	low C_3A	- Sulfate resistant

For complete hydration, cement requires a minimum amount of water equal to about 25 % of its weight. While it is desirable to minimize the amount of water in the mix, a water-cement ratio of about 0.30 is the lowest value that can be achieved in practice for a proper workability. The term "workability" means that the grout can easily be pumped to the required height. Water in excess of that used in the hydration process causes small voids in the resulting grout. This weakens the grout and makes it more porous. An optimum water/cement ratio is, however, required to achieve both grout strength and workability. An optimum water/cement ratio is selected which fulfills both functions i.e. (a) grout to reach everywhere and (b) grout should not flow and segregate under the force of gravity.

Natural pozzolans, fly ash, condensed silica fume and blast furnace slag are all supplementary cementitious materials which are sometimes used to partially replace

Portland cement or to enhance the characteristics of the resulting grout. Condensed silica fume or 'micro-silica', which is a by-product from the manufacture of ferro silicon, about two order of magnitude finer than Portland cement (Collins & Mitchell, 1987). When used to replace 5 to 10 per cent of the Portland cement, it can result in a very high strength concrete.

Superplasticizers are linear polymers containing sulfonic acid groups which temporarily increase the workability of the concrete. High strength cement requires a higher water/cement ratio because of silica fume, and to balance this, superplasticizers are added. The use of admixtures enables low water/cement ratios (0.3 to 0.4) to be used while maintaining workability, resulting in high strength and low permeability concrete or grout (Collins & Mitchell, 1987).

Water reducers are used to reduce the amount of water in grout. The main constituents are Lignosulphonic acids and their salts. The principal active components of the admixtures are surface-active agents. These are substances which are concentrated at the interface between two immiscible phases and which alter the physico-chemical forces acting at this interface. The particles have, therefore, a greater mobility and free from the restraining influence of the flocculated system, becomes available to lubricate the mix so that the workability is increased (Nevil, 1981).

3.3.2 Aggregate

Natural sand as fine aggregate and crushed stone as a coarse aggregate were used to obtain shotcrete grout. Aggregates occupy more than 50 percent of the total volume of the matrix, therefore, they have a definite influence on the mechanical properties of shotcrete grout. A high proportion of fine or coarse aggregate in concrete or shotcrete mixes produces undesirable and unsuitable results. Large quantities of coarse aggregate in a mix results in harshness, bleeding and segregation, whilst, excessive fine aggregate requires a comparatively large amount of water to produce the necessary fluidity; it also tends to cause segregation (Abrams and Orals, 1965). The sieve analysis of fine and coarse aggregates is given in Table 3.2.

Table 3.2: Sieve analysis of fine and coarse aggregates used in shotcrete mix

Sieve No.	Sieve Size	Percent of Passing	
U.S.	(mm)	Fine Agg.	Coarse Agg.
3/8	9.50	100	100
4	4.75	100	41
8	2.36	97	3
16	1.00	80	0
30	0.60	46	0
50	0.30	33	0
100	0.15	9	0

3.4 High Strength Cement Grout (HSCG) versus Conventional Cement Grout (CCG)

The basic purpose of introducing HSCG in the mining industry is to install the support system in a shorter time. Figure 3.1 and Table 3.3 show the time taken by HSCG & CCG to gain compressive strength. From the figure, it can be seen that HSCG achieves approximately 50 MPa in 2 days. The proportions of constituents of HSCG grout are given in Table 3.4.

Figure 3.2 shows stress-strain behaviour of both HSCG and CCG. The modulus of elasticity of HSCG does not increase linearly with the compressive strength and its maximum value ranges between 40 to 50 GPa. Consequently, the ductility of supported elements by HSCG will, more or less, remain the same as achieved in case of CCG.

Table 3.3: Ultimate compressive strength of High Strength Cement Grout (HSCG) and Conventional Cement Grout (CCG)

Cement type	Time (Days)	Ultimate strength (MPa)			Mean (MPa)
		Test Number			
		1	2	3	
CCG	1	12.58	12.72	12.14	12.48±0.3
HSCG	1	17.26	23.64	16.42	19.11±3.95
CCG	2	30.17	28.77	30.62	29.85±0.96
HSCG	2	50.0	48.33	49.16	49.16±0.84
CCG	3	32.8	33.31	34.1	33.4±0.66
HSCG	3	61.3	60.1	58.3	59.9±1.51
CCG	7	49.17	41.8	51.23	47.4±4.96
HSCG	7	71.24	76.2	80.45	75.96±4.61
CCG	14	50.56	48.15	51.0	49.9±1.53
HSCG	14	92.0	88.84	85.3	88.71±3.35
CCG	28	52.0	58.1	50.7	53.6±3.95
HSCG	28	97.46	96.5	95.2	96.39±1.13

Table 3.4: Representative mix proportion for HSCG

Components	Quantities
Cement	10 kg
Water	4 kg
Superplasticizers (Mulcoplast CH)	100 ml (10 ml/kg)
Water reducer (TCDA x AH)	25 ml (2.5 ml/kg)

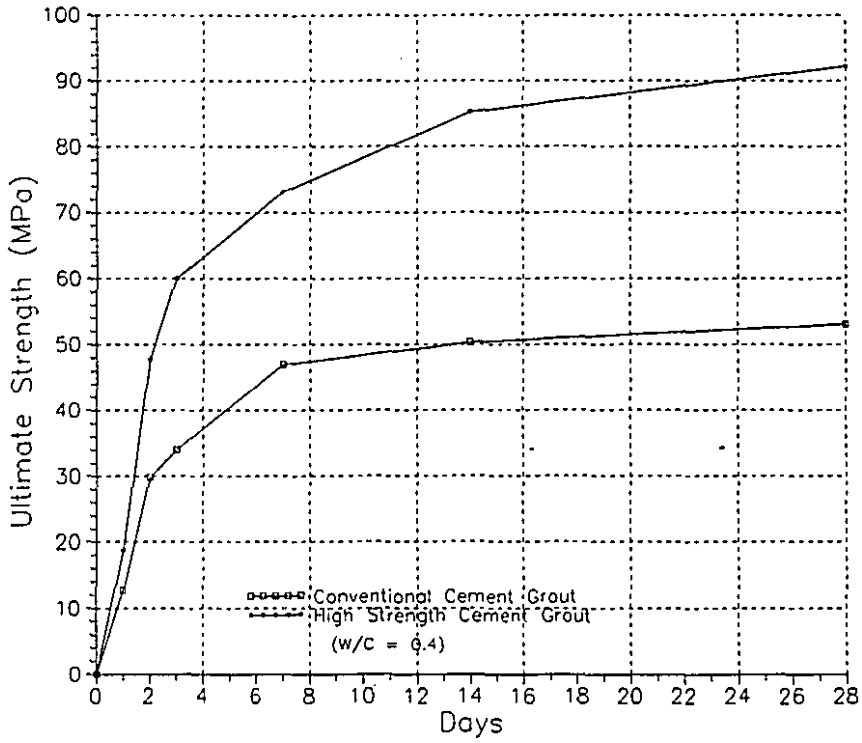


Figure 3.1: Compressive strength vs. time of HSCG and CCG

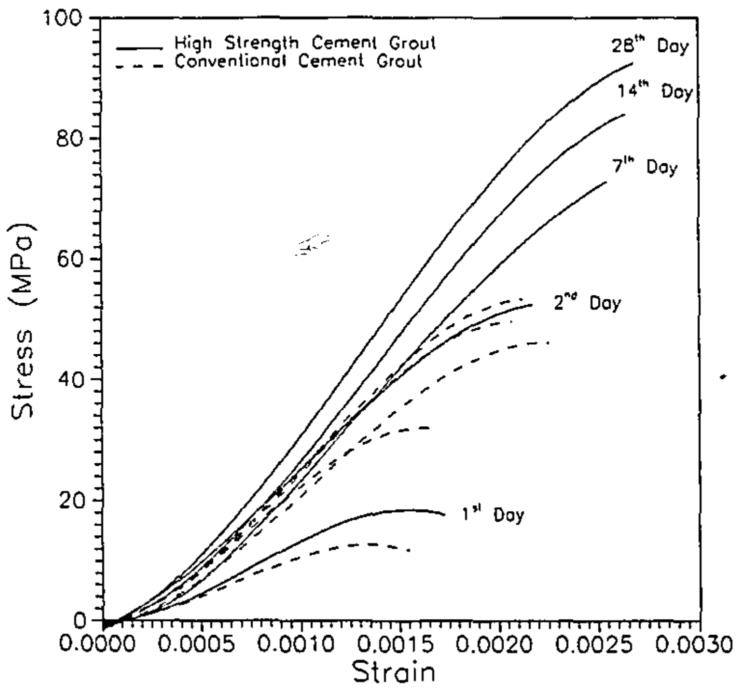


Figure 3.2: Stress strain behaviour of HSCG and CCG (Lines represent 1, 2, 7, 14 and 28 days of curing)

3.4.1 Effect of Confinement Pressure on HSCG and CCG

The purpose of this series of experiments was to investigate the ductile behaviour of both CCG and HSCG, and then compare their performance. The experimental procedure contained the following steps:

1. The preparation of small rectangular samples of CCG and HSCG. After two days of curing, a small strip with the dimensions: thickness = 5 mm, length = 20 mm and width = 12 mm was cut from the rectangular samples.
2. A strain gage, according to specified procedure, was glued (stuck) onto the surface of the strip.
3. This strain gage-attached-strip was fixed horizontally in the middle of the cylindrical mould by means of cotton threads.
4. Grout was poured slowly without disturbing the position of strain gage strip. These samples were stored for 28 days for curing.
5. This cylindrical grout sample was encased in a rubber tube and then inserted and fixed in Hoek's Triaxial Cell, as shown in Figure 3.3.
6. External confining pressure and corresponding strain variations in the embedded strain gage were recorded and results are graphically shown in Figure 3.4.

Both cements behaved slightly different under confinement pressure; see Figure 3.4. A linear relationship is observed between confinement pressure and micro strain; which develops within the body. On the other hand, the curve representing HSCG shows that the behaviour is stiffer in the low pressure range, however, cracks start developing under a higher pressure and cause a more ductile response. The load bearing capacity of HSCG is still maintained until complete failure.

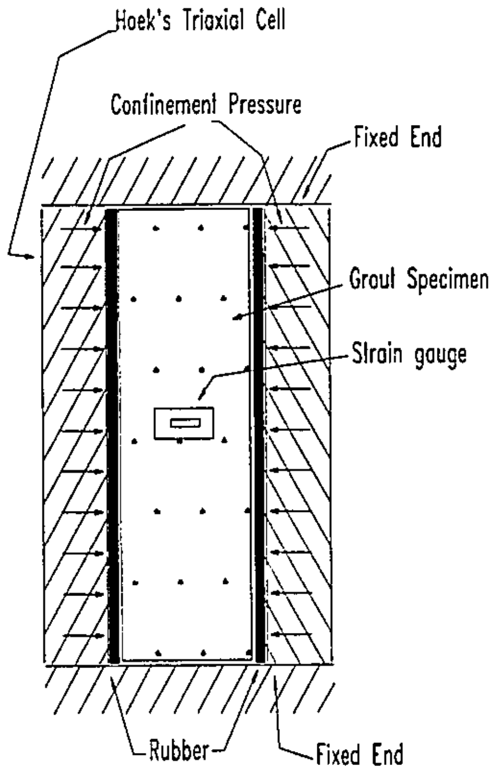


Figure 3.3: Hoek's Triaxial Test setup

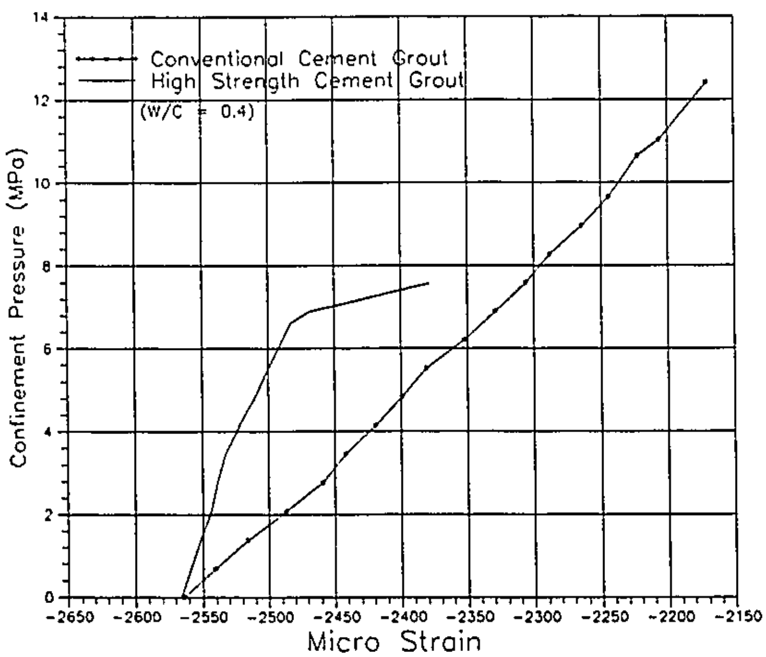


Figure 3.4: Behaviour of HSCG and CCG under confinement pressure

3.5 Shotcrete Grout

The term "shotcrete grout" means combination of coarse and fine aggregates with cement. Previous researchers such as Farah and Aref (1986) and Rajaie (1990) used shotcrete grout with both coarse and fine aggregates. It is believed that the presence of fine aggregate into a mix is to fill the gaps between the coarse aggregate and it does not contribute to the overall strength of the grout. Therefore, shotcrete grout samples with and without fine aggregate were prepared and tested. Additionally, combination of high strength cement and coarse aggregates were also tested. The mechanical properties of different types of grout are presented in Table 3.5

Table 3.5: Mechanical properties of different grouts

Grout Type	UCS (MPa)	E (GPa)	$\varepsilon_{\text{at failure}}$ (%)
Conventional Cement (CC)	53 ± 4.2	31 ± 3.3	0.23
High Strength Cement (HSC)	94 ± 6.2	44 ± 3.7	0.18
CC + Coarse and Fine Aggregates*	35 ± 5.1	26 ± 3.9	0.20
CC + Coarse Aggregates!	48 ± 5.5	25 ± 4.1	0.19
HSC + Coarse Aggregates!	88 ± 4.3	40 ± 2.8	0.22
Organic Resin	76 ± 2.1	11 ± 1.8	0.9

* The mix proportion was 1:1:1; ! The mix proportion was 1:1

3.6 Organic Resin Grouts

Polyester resin and epoxy resin are organic thermoset resins, which are widely used in mining applications. This particular type of resin is formed by an exothermic chemical reaction between the resin and a catalyst that acts as a hardening agent. Polyester resins are preferred over epoxy resins because of their lower cost and faster hardening rate at mining temperatures. The advantages and disadvantages of both cement and resin grouts are summarized by Choquet (1991)

An inherent disadvantage of polyester resins is that hardening is accompanied by shrinkage. Pure resin may shrink 8 % to 17 %, so that a filler consisting of quartz or calcite is generally used to reduce shrinkage to less than 1 %. This filler also makes it possible to reduce the unit cost of the material; however, it also reduces the natural ductility of the resin and makes it more brittle.

Manufactures (Celtite, Ground Control, Dupont) produce resins with setting times ranging from 1 to 30 minutes. However, the setting time varies with temperature, as shown in Figure 3.5. There is a wide variation in the physical and mechanical properties of resins because of the manufacturing process. Typical mechanical properties of resins are shown in Table 3.6. These properties may change considerably with temperature as shown in Figure 3.6.

Table 3.6: Mechanical properties of Polyester resin (Fastloc-T, Dupont)

Density (kg/m^3)	Tensile Strength (MPa)	Compressive Strength (MPa)	Shear Strength (MPa)	Modulus of elasticity (MPa)	Poisson's Ratio	Elongation at Rupture (%)
1.85	17	110	50	7000	0.3	0.2

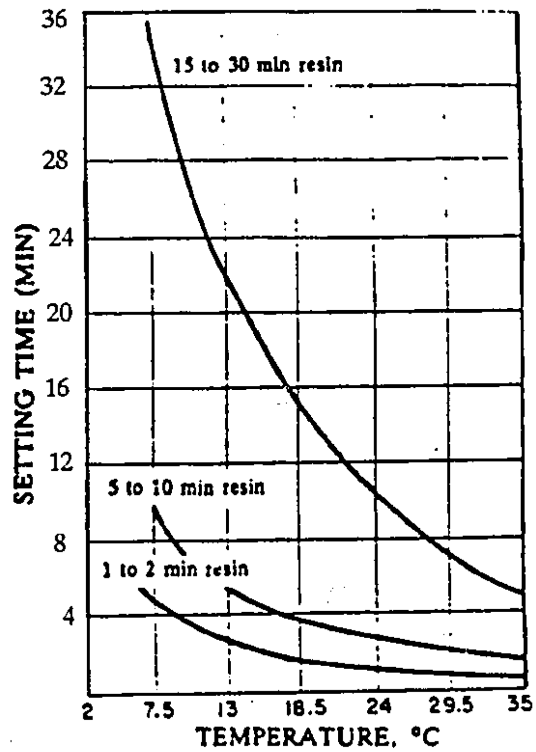


Figure 3.5: Effect of temperature on the setting time of Polyester resin (Fastloc-T, Dupont)

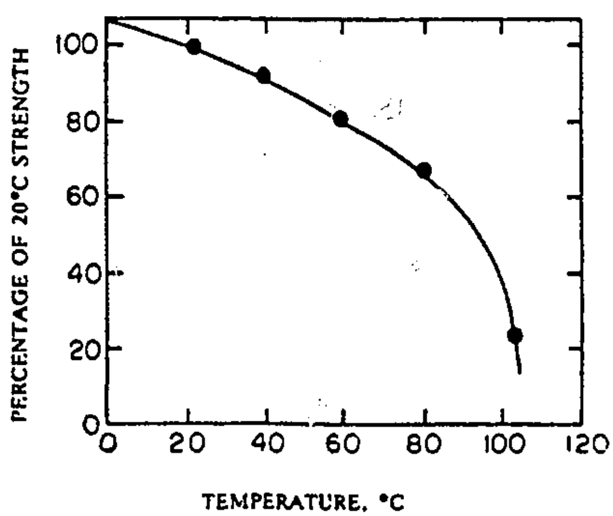


Figure 3.6: Effect of temperature on the compressive strength of Polyester resin (Beveridge, 1974)

3.6.1 Installation Method of Organic Grout

The installation method used with resin-grouted bolts varies very slightly from one location to another. The procedure is illustrated in Figure 3.7. It consists of introducing a certain number of resin cartridges in drill hole. The bolt is then installed using a jack leg drill or power wrench, while spinning the bolt to break the cartridges and mixes resin with its catalyst. The duration of the mixing stage should be neither too short nor too long. The resin manufacturer's guidelines for the installation should be followed. An incorrect mixing of resin can lower its properties. The diameter of the drill hole is also important and should be 6.35 mm larger than the diameter of the bolt. This diameter minimizes the amount of resin that must be used, while it ensures a good mixture and maximum holding power in comparison with bolt installed in drill holes with smaller or larger diameter. The diameter of the cartridge must be selected in such a way that it should loosely fit in the drilled hole and must fill the 3.2 mm space around the bolt barrel. For example, for a 19.05 mm bolt in a 25.4 mm hole, the required cartridge amount is calculated as the difference in cross-section between the bolt and the hole. The diameter of commercially available cartridges is generally little less than the hole, making it possible to install cartridges over a longer distance than the total length of the drill hole. The manufacturers provide tables that can be used to evaluate the grouted length as a function of the diameter of the bolt, the cartridge, and the drilled hole (Choquet, 1991).

3.6.2 Manually Mixable Polyester Resin

A new type of organic resin (polyester resin), which can be manually mixed in the laboratory, has been introduced in this research. This resin eliminates the process of spinning of bolt in order to mix resin cartridges in the drill hole. Therefore, pull out samples with small embedment lengths can successfully be prepared with this resin. The mixing procedure is simple, resin and catalyst are mixed thoroughly in a container and then poured into the drill hole (steel pipe or drill hole in concrete or rock block). Styrene, a catalyst, can be added to decrease the viscosity of resin; this makes resin easy to pour into the drill hole.

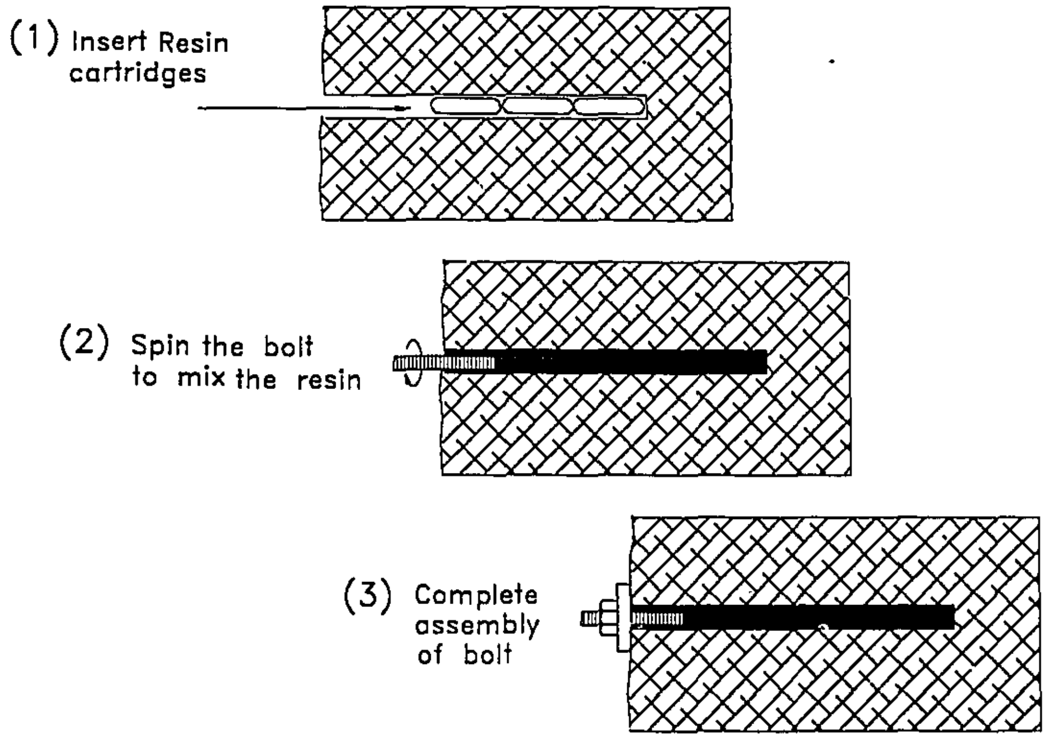


Figure 3.7: Installation of tendons with conventional resin cartridges

Mechanical properties of resin, which were experimentally evaluated, are tabulated in Table 3.7. The mix ratios of 1 to 13 Catalyst-Resin (weight pct) were used to prepare the mix. ISRM (International Society of Rock Mechanics) standards were followed to prepare samples. Stress-strain relations, obtained from uniaxial compressive strength, are graphically shown in Figure 3.8. The Brazilian test was conducted to obtain the tensile strength of resin. Maximum load at failure was recorded for each sample and then tensile strength was calculated from it.

Experimentally, Poisson's ratio was obtained by using a special type of LVDT, which can be fixed circumferentially on the cylindrical sample. Data acquisition system simultaneously recorded both axial and diametric strain. From the obtained data, strain of resin 'v' is calculated and presented in Table 3.7. The plot of axial strain versus diametric strain is also presented (see Figure 3.9) to compare the strain changes under gradually increasing loads. Shear modulus of resin 'G' is calculated as E/2∗(1+v).

Due to high viscosity of resin, it was difficult to prepare samples, therefore chemical catalyst known as Styrene, was added to the mix. Different percentages by weight of Styrene were tried and eventually 5 % Styrene was recognized to be the appropriate value for the mix. Stress-Strain diagrams for all percentages of Styrene, tested in the laboratory, are plotted in Figure 3.10. It can be seen that Styrene has no effect on the mechanical properties of resin and the gradients of stress-strain curves remain unchanged.

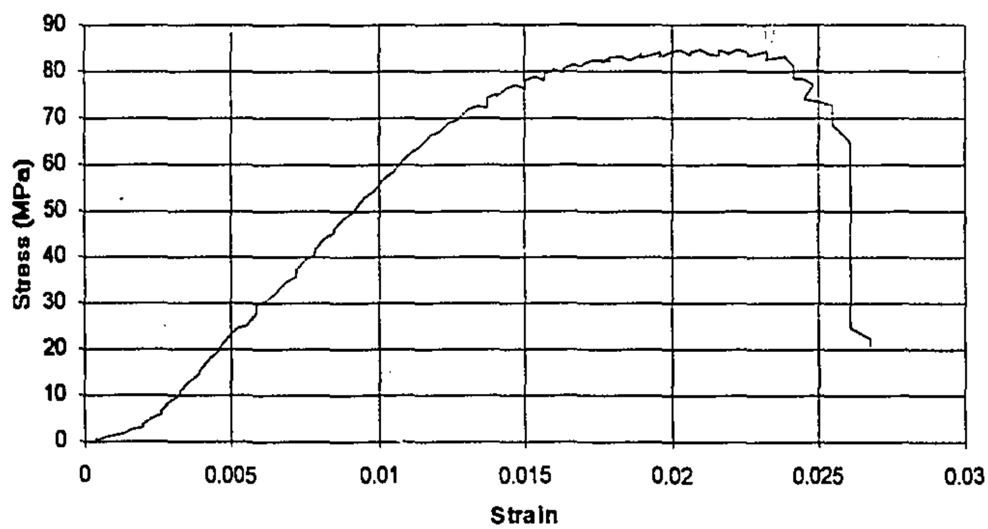


Figure 3.8: Stress- strain diagram for resin.

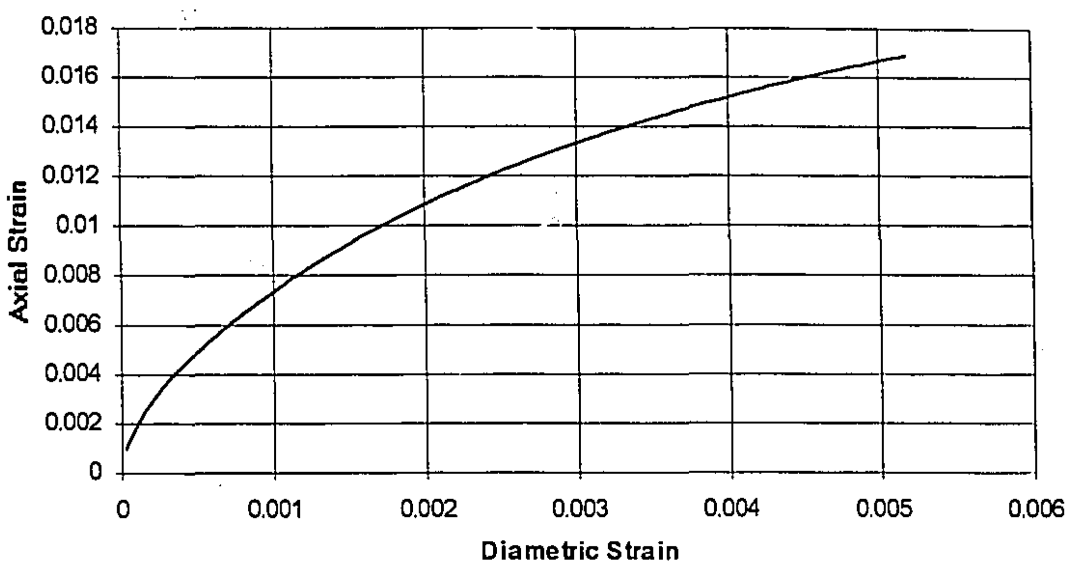


Figure 3.9: Axial strain versus diametric strain.

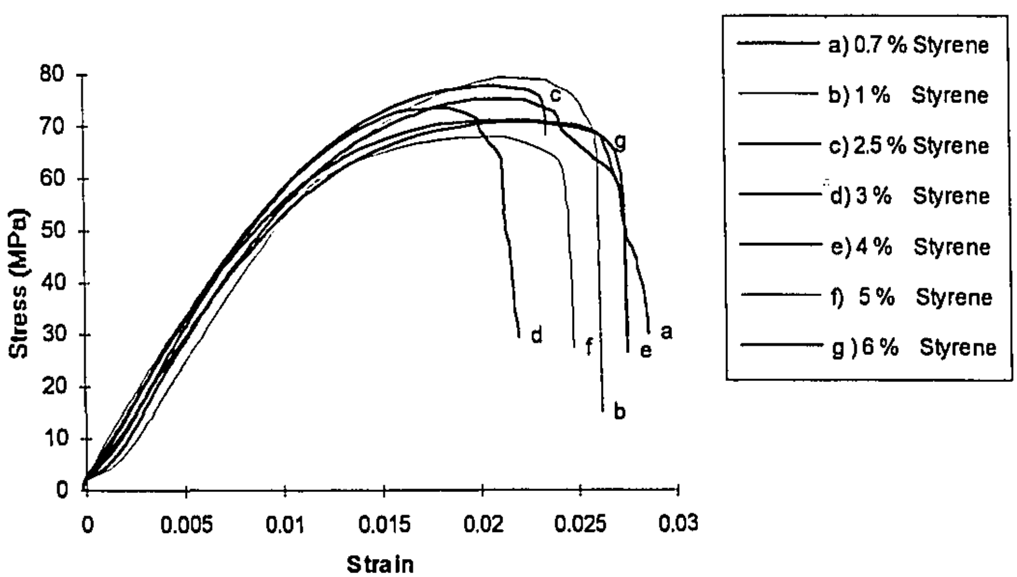


Figure 3.10: Effect of Styrene on the behaviour of Resin.

Table 3.7: Mechanical properties of resin.

Property	No. of Samples	Mean Value
Uniaxial Compressive Strength (MPa)	10	75.72 ± 0.06
Tensile Strength (MPa)	3	8.47 ± 0.0099
Modulus of Elasticity (GPa)	11	5.56 ±0 .093
Shear Modulus (GPa)	11	2.33 ± 0.09.4
Axial strain at 20 % of ultimate strength	3	0.0022
Axial strain at 50 % of ultimate strength	3	0.0058
Axial strain at 80 % of ultimate strength	3	0.0089
Dia. strain at 20 % of ultimate strength	3	0.00019
Dia. strain at 50 % of ultimate strength	3	0.00066
Dia. strain at 80 % of ultimate strength	3	0.00014
Poisson's Ratio	3	0.19 ± 0.072
Density (gm/cm^3)	3	2.10 ± 0.0082

3.7 Discussion

The following conclusions can be drawn from the experimental results:

- High strength cement grout (HSCG) is found superior in performance over conventional cement grout (CCG). The uniaxial compressive strength (UCS) was obtained around 90 MPa for former and 50 MPa for latter. The modulus of elasticity of HSCG, however, did not increase linearly with UCS and found between 40 - 50

GPa. Whereas in CCG, this value ranged between 25 - 35 GPa. These results ensured the ductile behaviour of HSCG.

- The other advantage of HSCG was its increase in compressive strength with time. After two days, around 50 MPa of UCS was obtained, however, same value for CCG was obtained after 28 days. This means that HSCG can be used for relatively fast installation of fully grouted cables.
- Addition of coarse or fine aggregates in cement (shotcrete grout) did not seem to increase its mechanical properties. Experiments show that fine aggregates need not to add to shotcrete mix, since, it made mixing troublesome and decreased the workability. The presence of coarse aggregates in cement grout is believed to increase the frictional resistance between cable and grout, during the pull out process.
- A manually mixable resin was introduced and tested, for the first time, to obtain its mechanical properties. The advantage of this system over conventional sausage (cartridge) grout is that it does not require spinning of bolt. Therefore, this grout can be used with good quality control for small embedment lengths. The uniaxial compressive strength and modulus of elasticity were found, respectively, 76 MPa and 11 GPa.

Chapter 4

Laboratory Investigation on Cable Bolt Supports

4.1 Conventional Cable Bolt Support

In general a cable bolt support system may be defined as long flexible steel strands which are fed into a borehole and fixed into the borehole by grout or resin. This could be installed in the form of passive or active support prior to or after excavation. A cable bolt support may be categorized as:

1. Support element (cable bolt) which consists of different shapes and types of steel strands,
2. Bonding element (grout) which consists of cement + water and/or additives, and
3. Host medium (rock mass) which consists of geological materials with inherent discontinuities at different stress regimes.

4.2 Testing Program

The following parameters were considered to be of prime importance in controlling the behaviour of the cable bolt and it was intended that their effects should be investigated in this research:

1. Hole diameter of loading plate
2. Stiffness and stress variations

(b) stiffness and strength of grout

(c) Variations in the induced stresses on grout-cable interface

3. Poisson's Ratio and Axial Strain of Rocks
4. Rate of displacement
5. Water cement ratio
6. Cable embedment length
7. Curing time
8. High strength cement grout
9. Shotcrete grout
10. Cuttable cable bolts
11. Shear resisting capacity of fully grouted cables

Three samples of each parameter were prepared and tested after 28 days of curing. The explanations of range of parametric values are the following:

- Hole Diameter of loading plate

  - Host medium used = three concrete blocks for each type
  - **Hole diameter (variable) = 20, 51 and 127 mm**
  - Embedment length = 240 mm
  - Grout = CCG; w/c = 0.4
  - Rate of loading = 1 mm/min

- Stiffness and stress variations

  (a) Stiffness of host mediums

  - **Host mediums (variable) = Schedule 80 steel pipe (ID = 57 mm), Weak and strong concrete blocks (ID = 50 mm), PVC pipe (ID = 50 mm)**
  - Embedment length = 240 mm
  - Grout = CCG; w/c = 0.4
  - Rate of loading = 1 mm/min

- Grout = CCG; w/c = 0.4
- Rate of loading = 1 mm/min

(b) Stiffness and strength of grout

- Host medium = Rock block (Charnockite, Gneiss Granite and Gabbro)
- Embedment length = 200 mm
- **Grout (variable) = CCG and HSCG + Coarse aggregates; w/c = 0.4**
- Hole diameter = 57.2
- Rate of loading = 1 mm/min

(c) Variations in the radial stresses

- Host medium = Hoek's Triaxial Cell to vary the radial stresses
- **Radial stresses (variable) = 2, 5, 7 and 10 MPa**
- Embedment length = 137 mm
- Grout = CCG; w/c = 0.4
- Grout column dia = 5.5 mm
- Rate of loading = 1 mm/min

• Poisson's Ratio and Axial Strain of Rocks

- Host medium = Rock block (Charnockite, Gneiss Granite and Gabbro)
- **Variables = Poisson's ratio and axial strains of rocks**
- Embedment length = 200 mm
- Grout = CCG and HSCG + Coarse aggregates; w/c = 0.4
- Hole diameter = 57.2
- Rate of loading = 1 mm/min

• Rate of displacement

- Host medium used = Schedule 40 steel pipe
- **Rate of loading (variable) = 1 cm/sec, 1 cm/min, 1 mm/min, 0.1 mm/min**
- Hole diameter = 20, 51 and 127 mm
- Embedment length = 305 mm
- Grout = CCG; w/c = 0.4

- Water cement ratio
  - Host medium used = Schedule 40 steel pipe
  - Hole diameter = 57 mm
  - **Water cement ratio (variable) = 0.3, 0.35, and 0.45**
  - **Embedment length (variable) = 200, 400 and 600 mm**
  - Grout = CCG
  - Rate of loading = 1 mm/min

- Cable embedment length
  - Host medium used = Schedule 40 steel pipe
  - Hole diameter = 57 mm
  - **Embedment length (variable) =150, 200, 300, 400, 500, 600, 750 mm**
  - Grout = CCG; w/c = 0.4
  - Rate of loading = 1 mm/min

- Curing time
  - Host medium used = Schedule 40 steel pipe
  - Hole diameter = 57 mm
  - Embedment length = 750 mm
  - Grout = CCG; w/c = 0.4
  - **Curing time (variable) = 1, 3, 7, 28 days**
  - Rate of loading = 1 mm/min

- High strength cement grout
  - Host medium used = Schedule 40 steel pipe
  - Hole diameter = 57 mm
  - Embedment length = 305 mm
  - **Grout (variable) = HSCG and CCG; w/c = 0.4**
  - Rate of loading = 1 mm/min

- <u>Shotcrete grout</u>
  - Host medium used = Schedule 40 steel pipe
  - Hole diameter = 57 mm
  - Embedment length = 305 mm
  - **Grout (variable) = Shotcrete grout with and without fine aggregates and CCG**
  - water cement ratio = 0.4
  - Rate of loading = 1 mm/min

- <u>Cuttable cable bolts</u>
  - Host medium used = Schedule 40 steel pipe
  - Hole diameter = 57 mm
  - Embedment length = 305 mm
  - **Cable type (variable) = Steel cable, Arapree cable and Polystal birdcage cable**
  - Grout = CCG; w/c = 0.4
  - Rate of loading = 1 mm/min

- <u>Shear capacity of fully grouted cables</u>
  - Host medium used = Two schedule 40 steel pipes
  - Hole diameter = 57 mm
  - Embedment length = 305 mm
  - **Joint spacing (variable) = 10 mm, 20 mm, 40 mm, 50 mm, 80 mm and 100 mm**
  - Grout = CCG; w/c = 0.4
  - Rate of loading = 1 mm/min

4.3 Testing Arrangements

A simple yet practical test set-up was devised to perform pull out tests on cable bolts. The testing arrangements were designed to accommodate variation of a wide range of parameters, such as different diameters of cable, tendons holding devices, tendons embedment lengths and different types of grouts (cement and resin). All the basic parameters which can influence the performance of conventional steel cable bolts were evaluated. Various researchers conducted pull out tests to obtain the behaviour of cable bolts. Some of these are listed below:

Fuller (1978) investigated the behaviour of a single wire and cable bolt by means of pull out tests. Two grout columns with different lengths were formed within two steel tubes. These tubes were pulled away considering the eventual shear bond failure to the smaller embedded length of the grout cable section and, also, this technique prevented the rotation of the cable with respect to the grout.

Stillborg (1984) investigated the behaviour of cable bolts by means of the pull out test. His typical pull out test was consist of a cable fixed into the cylindrical concrete block by the applications of cement grout. The cable was pulled out once the grout was cured. A steel grip or chuck was used to hold the cable during pull out tests. However, no measures were taken to prevent the rotation of the cable.

Goris (1987) demonstrated the characteristics of the different configurations of cable bolts by the testing arrangements, which were almost similar to Fuller's, except that in order to prevent slippage of the end of cable, a steel washer was welded to the one end face of steel strands. The displacement of the cable was monitored by two potentiometers attached to the pull out test sample.

A technique similar to Stillborg's has been adopted herein. Additionally, special arrangements were made to prevent the rotation of cable with respect to the grout during the pull out tests. The experimental arrangements undertaken for composite tendons were slightly different from conventional steel cables, therefore, their test arrangements are explained in their specific sections. A minimum of three samples was prepared for each category of testing program. One test result, which was close to the average of all three, category of testing program. One test result, which was close to the average of all three, was selected for the graphical representation. The detailed data is tabulated along with the graphs to present the average values of the experiments. The graphical average (of three or more tests) was not calculated, since it conceals the real behaviour of pull out tests (such as chemical adhesion and elastic behaviour prior to peak-failure and stick-slip post-failure).

4.3.1 Sample Preparation

The following procedure was employed for sample preparation; refer to Figure 4.1:

a) The cable was cleaned and cut to the desired length.
b) The host medium was modelled by a concrete block, a rock block or a schedule 40 steel pipe, depending on the nature of the experiment. Unless otherwise specified the "borehole" diameter of concrete blocks is 50 mm and embedded length is 240 mm, and in the case of steel pipes, the inner diameter is 57 mm and the length is 305 mm.
c) The cable is placed at the center of the borehole and is extended outside the borehole approximately 12 cm at the free end and approximately 15 cm at the loaded end.
d) Cement grout with water cement ratio of 0.4 is prepared, mixed and then poured into the borehole.
e) Each specimen was stored in the curing room at a temperature range of $20 \pm 3^{o}C$, in a highly moist atmosphere during the curing period (roughly 28 days).

Extensive laboratory study was launched to investigate various mixes of grout in order to optimize the support capability of cable bolting system. Over 300 pull out tests were conducted to investigate various parameters controlling the bond strength of the cable bolt.

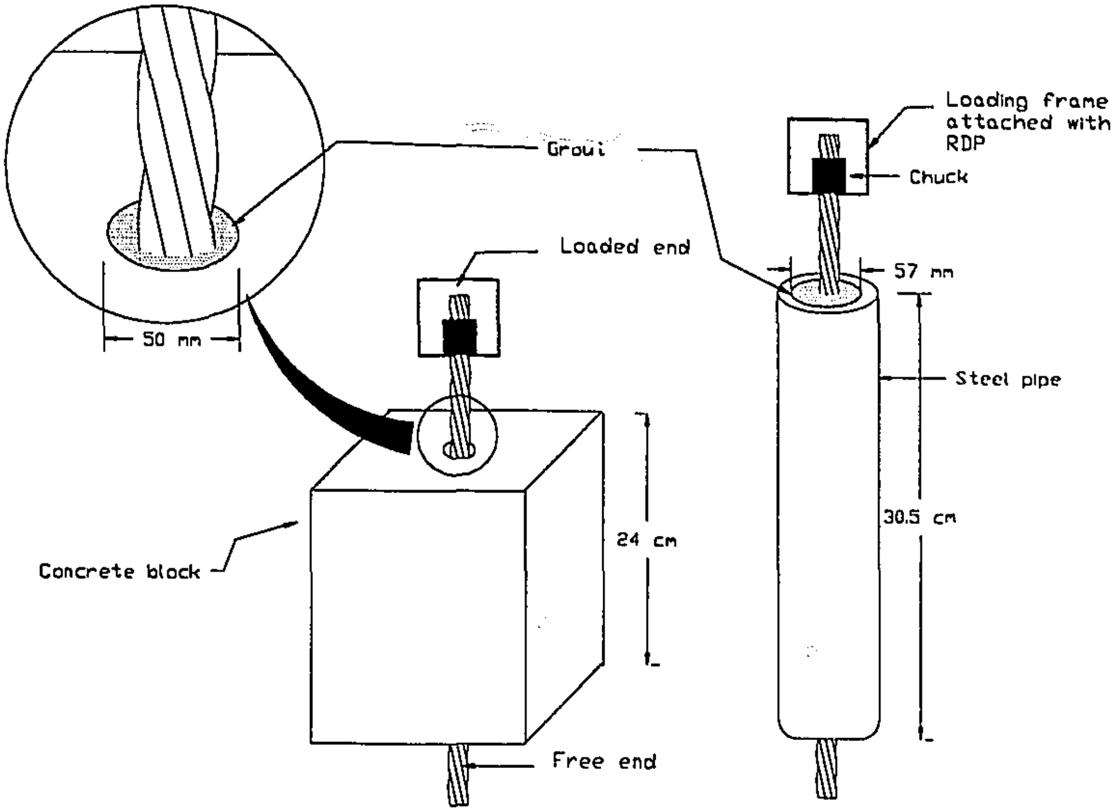


Figure 4.1: Description of samples for pull out experiments

Figure 4.2: Laboratory testing arrangements

R.D.P Howden servo controlled stiff testing machine with a capacity of 2500 kN in compression and 1250 kN in tension was used in the experiments. The tests were conducted under constant displacement control mode. The rate of displacement increments was selected to be small enough (i.e. 1 mm/min) to reflect the non-linearity of the response of bond stress between cable and surrounding matrix. The number of increments was sufficiently large to provide approximately 100 mm displacement for each test. A stiff steel frame was used to adjust and to hold the pull out samples in the R.D.P. Howden testing machine. The load and displacement were measured directly by load and displacement transducers, respectively, in order to monitor the slip between cable and surrounding matrix. LVDT (Linear Voltage Differential Transformer) was also used parallel to the automatic data acquisition system to verify the results. An Apple II computer was used to record the data, which was further saved on the floppy disks.

4.3.2 Problems and Sources of Error

Pull out tests on steel cables and bolts have been conducted in the Rock Mechanics Laboratory of McGill University by author's contemporaries (Rajaie, 1990). Therefore, the RDP Serve Controlled Stiff Testing Machine and data acquisition system have been repeatedly checked for accuracy. For present series of experiments, a different type of steel frame was designed and manufactured (see Figures 4.1 and 4.2). Some of the previously conducted experiments were repeated and it was found that the obtained results were in good agreement with previous results. No major difficulties or problems were encountered during the conventional steel cable pull out experiments. Some problems were faced due to incompatible gripping devices of composite tendons and there are discussed in Section 7.2.1.

The following were the sources of error and their effects on results:

1. It was not found easy to constraint the rotation of cable during pull out tests. Cable started to "un-screw" itself during pull out process and this might have reduced its pull out capacity. In order to constraint rotation, a special sliding box was attached with the piston of RDP Howden servo controlled stiff testing machine. The mechanism was that during the pull out process, when the box will rotate slightly due to "un-screwing" force of cable, it would come in contact with sliding box and then onwards rotation will be constrained. This arrangement controlled the rotation to great extent, however, some sort of rotation was induced in the system in the process of testing frame to come in contact with sliding box. This rotation was neither could be measured nor could be deducted from the obtained data. This problem was, however, not encountered when two-steel-pipe technique was used in the pull out tests; see Section 7.5.1.
2. When the pull out frame came in contact with sliding box, due to friction some 5 - 7 kN load was lost. This drop occurred only when the pull out load was above 50 kN. This loss of load was found insignificant on the overall pull out behaviour.
3. Gripping devices were adjusted in such a manner so that they should not slip during the experiments. Some of them, however, did slip, but this slip was deducted and measured by means of LVDT, applied at the bottom of the pull out cable sample. This slip was later on deducted from the obtained results.
4. In the shear resisting experiments of steel cable, in few tests, the shear box attached to the piston, was titled on one side during the experiments. This happened in the post peak stage of experiments. This "tilt" could not be measured and therefore could not be deducted from the obtained results.

4.4 Experimental Evaluation of Various Design Parameters

In the following sections, each parameter is defined along with its experimental nature, pull out behaviour and its implementations in the field:

4.4.1 Hole Diameter of Loading Plate

In laboratory experiment, the loading plate diameter is often considered unimportant but the test results have shown that a proper loading plate diameter is essential. The pull out capacity is underestimated if loading plate diameter is too small (20 mm i.e. a little bigger than cable), or very big (127 mm). A 50 mm loading plate diameter is suitable to model pull out experiments. The test results are shown in Figure 4.3 and apparatus set up of experiment is shown in Figure 4.4.

A small hole diameter applies the resultant of axial load of cable mostly on the cable-grout interface only rather then distributing over the entire area. A proper distribution of forces is resulted in a hole diameter of 51 mm, where a larger diameter most probably forces the grout to dilate by stretching away the rock mass away form the cable. In this process, the effect of radial confinement is minimized and therefore, a premature failure is resulted. The application of this type of testing is, however, limited to laboratory set up only.

Table 4.1: Experimental values of effect of hole diameter of loading plate

Hole Dia (mm)	Peak Load (kN)				Average Residual Load (kN)			
	Test Number				Test Number			
	1	2	3	Mean	1	2	3	Mean
20	28.4	30.2	26.5	28.37±1.87	31.5	32	28.9	30.8±1.66
51	50	47.3	53.5	50.27±3.11	50	51.4	50.2	50.5±0.76
127	34.8	32.2	35.8	34.27±1.86	36.2	35.8	38.1	36.7±1.23

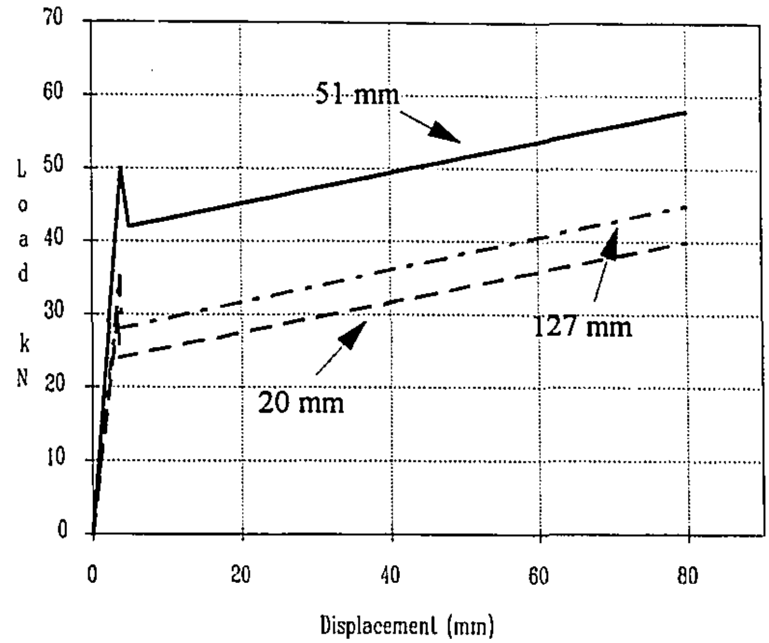


Figure 4.3: Effect of hole diameter of loading plate

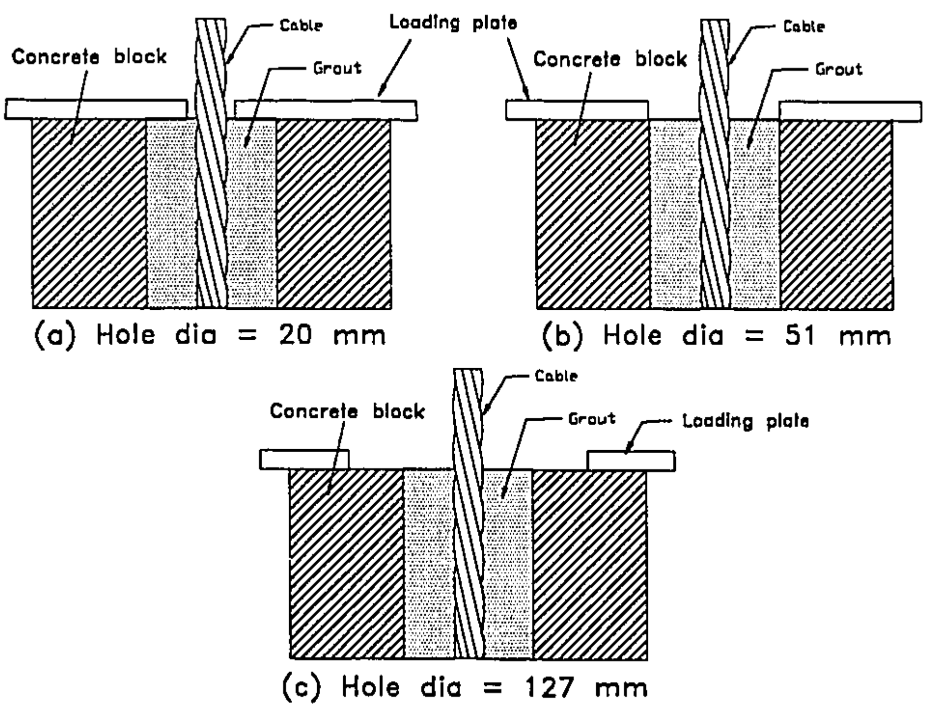


Figure 4.4: Different hole diameters of loading plate

4.4.2 Stiffness and Stress Variations

The following are the parameters investigated herein:

1. Stiffness of host medium
2. Stiffness and strength of grout
3. Variations in the induced stresses on grout-cable interface

Stiffness of Host Medium

Different moduli of elasticity of rocks are modelled by considering steel pipe, concrete blocks of modulus as 12 GPa and 2 GPa and PVC pipe. After curing for 28 days, PVC pipe was removed from its samples to model a zero modulus. Laboratory experiments were conducted and results are shown in Figure 4.5, which reveals two important facts that:

(a) The capacity of the cable bolt becomes nominal after peak-load in case of weak concrete block (E = 2.1 GPa) or PVC pipe (E = 0 GPa). This confirms that in the absence of high stiffness, cracks keep on propagating after reaching the elastic limit of grout.

(b) Results show that once the stiffness of host medium exceeds 10 GPa, the performance of cable support element does not improve on further increase in host medium stiffness. The only meaningful effect is on the residual supporting capacity; which keeps on increasing after peak failure.

Table 4.2: Capacity of fully grouted cable in different boundary conditions

Host Medium	Peak Load (kN)				Average Residual Load (kN)			
	Test Number			Mean	Test Number			Mean
	1	2	3		1	2	3	
Steel	53.4	55.2	50.8	53.1±2.21	62.5	65.0	59.3	62.3±2.86
Concrete block	50.0	51.0	48.5	49.8±1.26	50	53.1	51.1	51.4±1.57
Weak block	28.6	30.3	26.0	28.3±2.17	14.3	16.8	14.2	15.1±1.47
PVC pipe	18.4	18.0	19.1	18.5 ±0.56	8.1	7.9	8.4	8.13 ±0.25

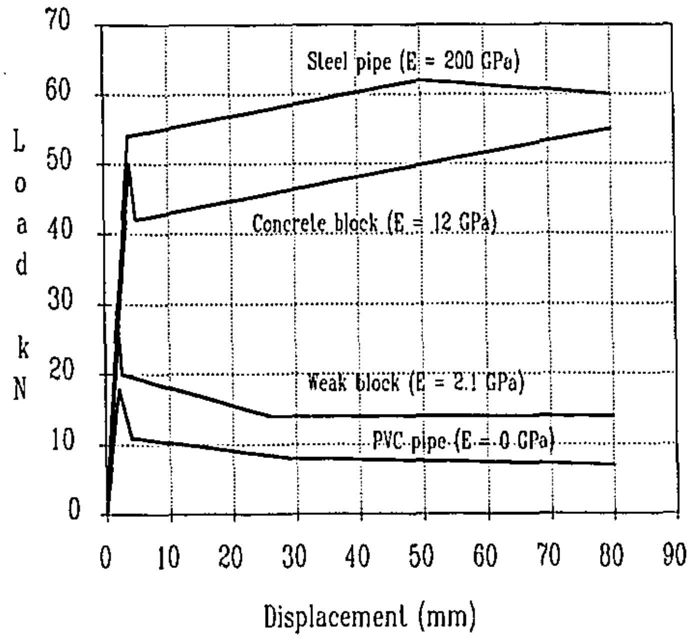


Figure 4.5: Effect of modulus of elasticity on cable pull-out resistance

Stiffness and Strength of Grout

High and low strength grouts were used to grout the cable in different types of rock to investigate the effect of grout (the mechanical properties of grouts are given in Chapter 3). Figure 4.6, 4.7 and 4.8 shows graphically the pull out results. From these graphs, it is obvious that stiff grout not only increases the capacity but also increases the post failure capacity i.e. residual strength. It also concludes that in an event of failure, it is likely that a detached rock will take more time to slip/fall when it is supported with a cable grouted with stiffer grout.

Table 4.3: Effect of varying stiffness of grout with Charnockite rock as host medium

Grout Type	Peak Load (kN)				Average residual load (kN)			
	Test Number			Mean	Test Number			Mean
	1	2	3		1	2	3	
HSC +CA	53.4	55.2	50.8	53.13±2.21	62.5	65.0	59.3	62.27±2.86
Con. Gr.	50.0	51.0	48.5	49.83±1.26	50	53.1	51.1	51.40±1.57

HSC + CA (Shotcrete) means High Strength Cement + Coarse Aggregates
Con. Gr. means Conventional Grout

Table 4.4: Effect of varying stiffness of grout with Gneiss Granite rock as host medium

Grout Type	Peak Load (kN)				Average residual load (kN)			
	Test Number			Mean	Test Number			Mean
	1	2	3		1	2	3	
HSC +CA	25.0	28.4	22.3	25.23±3.06	27.0	30.0	22.0	26.33±4.04
Con. Gr.	12.2	14.9	11.0	12.70±2.00	14.0	14.2	13.8	14.00±0.20

HSC + CA (Shotcrete) means High Strength Cement + Coarse Aggregates
Con. Gr. means Conventional Grout

Table 4.5: Effect of varying stiffness of grout and Gabbro rock as host medium

Grout Type	Peak Load (kN)				Average residual load (kN)			
	Test Number			Mean	Test Number			Mean
	1	2	3		1	2	3	
HSC +CA	36.7	39.0	33.4	36.37±2.81	51.0	54.0	48.0	51.00±3.00
Con. Gr.	19.0	21.0	17.4	19.13±1.80	22.0	21.0	22.0	21.67±0.58

HSC + CA (Shotcrete) means High Strength Cement + Coarse Aggregates
Con. Gr. means Conventional Grout

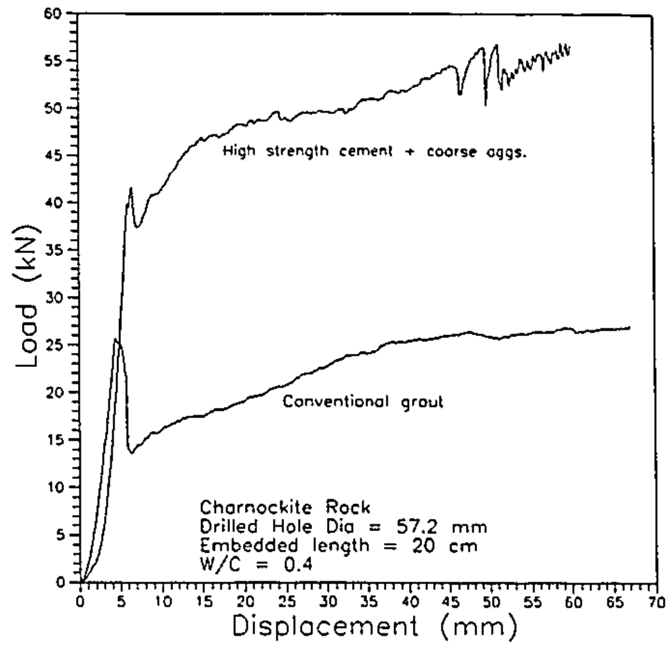


Figure 4.6: The effect of varying stiffness of grout and host medium (Charnockite rock)

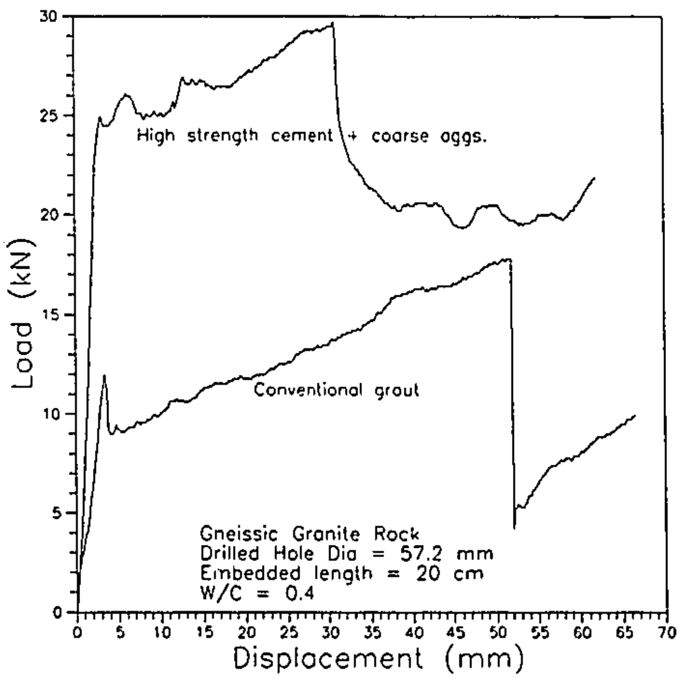


Figure 4.7: The effect of varying stiffness of grout and host medium (Gneissic Granite rock)

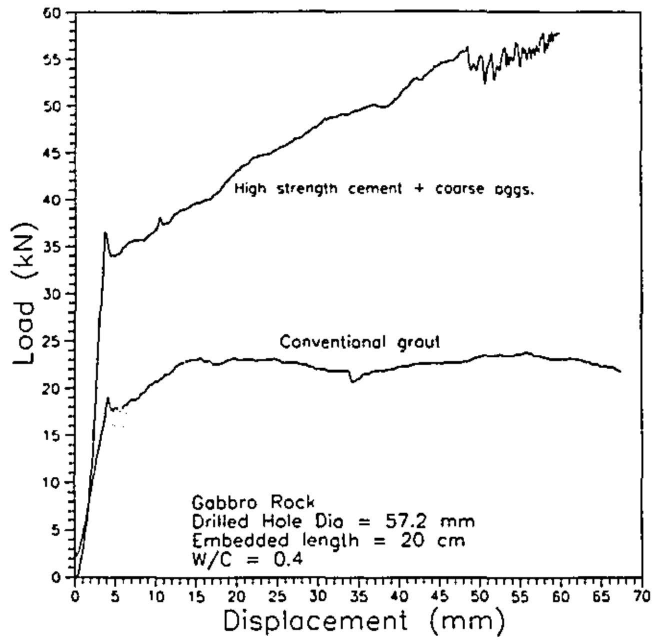


Figure 4.8: The effect of varying stiffness of grout and host medium (Gabbro rock)

Variations in the Radial Stresses

Various mining activities close to an area supported by fully grouted cable bolts can increase or decrease mining stresses in the components of support such as in cable element and/or at cable-grout interface, etc. In order to evaluate the performance of supporting element, cables were pulled out under varying confining pressures. Cables were embedded in PVC pipes (diameter = 51.5 mm, embedded length = 137 mm) and on curing of grout, the PVC covering was removed and grouted portion was inserted in Hoek's Triaxial Cell. Cables were pulled out while radial pressure was exerted on the grout (see Figure 4.9). Two different diameters of seven wire strand were used to see the effect of grout column diameter around the cable. Figure 4.10 and 4.11 represents graphically the test results. The following conclusions can be drawn from the figures:

1. There is always confinement due to the stiffness of rocks. Confinement cannot be zero.

2. Figures also show that there is not appreciable increase in load sustaining capacity of cable after 2 MPa of confinement. A confinement of 5 MPa or 7 MPa on the cable grout interface is unrealistic and cannot be found in underground mining (personal communication).
3. If the confinement pressure is more than the tensile strength of grout, then an adverse effect should be anticipated. A confinement of 10 MPa fractured (crushed) the grout column (Figure 4.10; curve with C=10 MPa).
4. In hard rock mining, effect of high confinement should be considered in mine planning and rock support design. In soft rock mining, however, thorough investigation of this particular parameter is also necessary prior to any design decisions.
5. The grout column diameter effects the pull out capacity of grouted cable. Thickness of grout between cable and Hoek's Cell (host medium) reduced the pull out capacity as compared to thinner grout. The possible explanation is that a large amount of radial stresses dissipates within the thick grout column and fewer stresses will transfer to the grout containing walls (host medium). In thin grout, comparatively more radial stresses are transferred to the rock walls, and hence, cable capacity is also controlled by wall strength and stiffness.

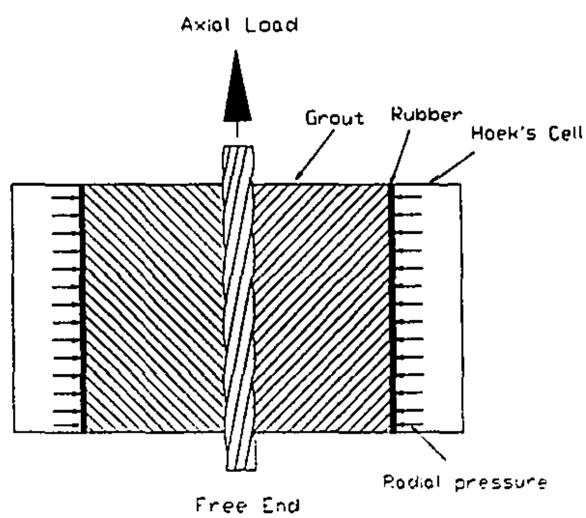


Figure 4.9: Grouted cable and Hoek's Triaxial Cell set-up

Table 4.6: Various confining pressure with cable diameter = 9.5 mm

Confining Pressure (MPa)	Average Residual Load (kN)			
	Test Number			Mean
	1	2	3	
2	25	33	21	26.33±6.11
5	30	37	28	31.67±4.73
10	**	**	**	**

** Sample crushed radially under 10 MPa confining pressure.

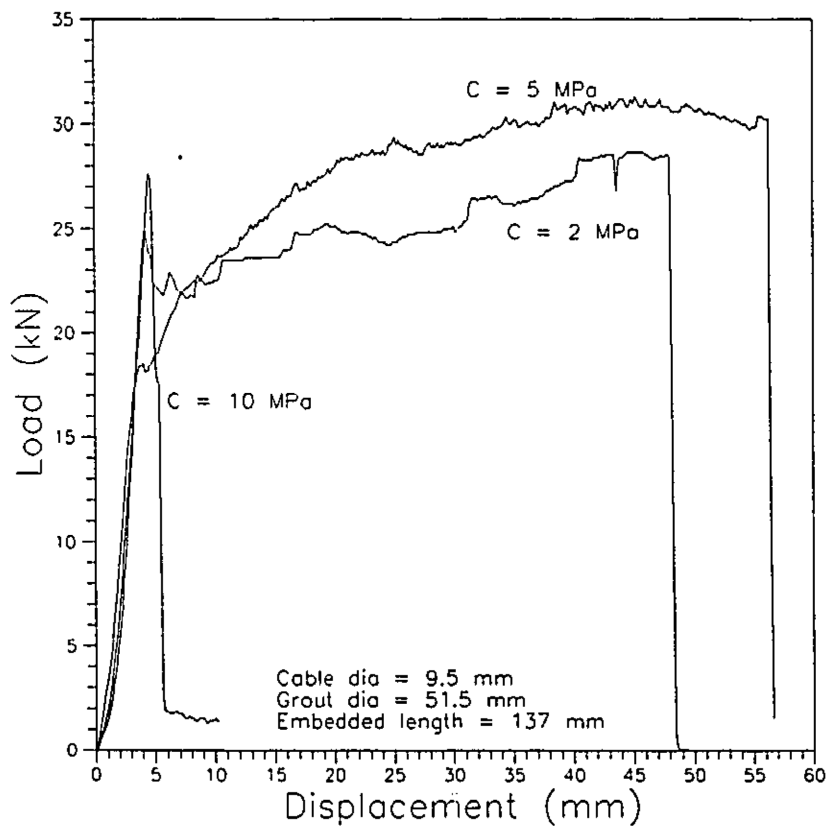


Figure 4.10: Varying confinement pressures and cable pull out capacity (cable dia = 9.5 mm)

Table 4.7: Various confining pressure with cable diameter = 12.7 mm

Confining Pressure (MPa)	Average Residual Load (kN)			
	Test Number			Mean
	1	2	3	
2	53	50	55	52.67±2.52
5	68	73	66	69.00±3.61
7	69	72	66	69.00±3.00

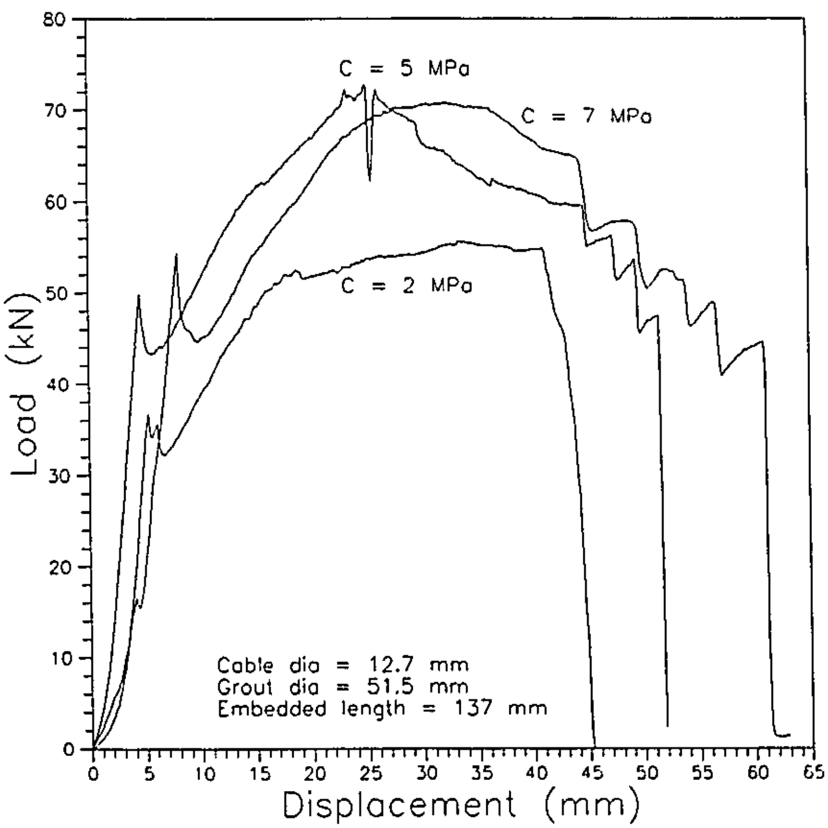


Figure 4.11: Effect of varying confinement pressure on the pull out capacity of cable (cable dia = 12.7 mm)

4.4.3 Poisson's Ratio and Axial Strain of Rocks

Three types of rock were used in this series of experiments to see the effect of the mechanical properties of rock, such as Poisson's ratio and axial strain, on the pull out capacity of fully grouted steel cables. The following were the types of rock used in the experiment:

- Gneissic Granite
- Gabbro
- Charnockite

A brief description of these rocks follows:

Gneissic Granite is an equigranular, hollocrystaline pink granite with gneissosity, having 35 % quartz, 35 % K-feldspar, 25 % plagioclase and 5 % biotite. The grains are 2 - 5 mm in diameter and the rock exhibits a sugary texture.

Gabbro is inequigranular, hollocrystaline pyroxene hornblende gabbronorite. It is made of 72 % hornblende crystals of 0.5 to 1.5 cm in size, 20 % pyroxene (3 - 5 mm), 3 % muscovite, 2 % opaques and 3 % alterations, mainly clay minerals.

Charnockite is an inequigranular and hollocrystaline rock made of 40 % K-feldspar (0.5 - 2 cm), 25 % quartz (2 - 9 mm), 30 % pyroxene (0.5 cm) and 5 % biotite (2 - 5 mm).

Along with the grout experiments, the effect of the mechanical properties of rocks on pull out capacity of fully grouted cables was also observed. Preparation of experiments comprised the cutting into cubes with each side 200 mm long. Four samples of each type of rock were prepared, three for pull out tests. The fourth was used to drill out cores to study the mechanical properties of rocks; which are tabulated in Table 4.8. Cables were grouted in the center of the rock cubes and tested after 28 days of curing.

Table 4.8: Mechanical properties of rocks

Rock Type	No. of Tests	Uniaxial Compressive Strength	Modulus of Elasticity	Strain at Failure	Poisson's Ratio
		MPa	GPa	%	
Gneissic Granite	3	204 ± 4	62.5 ± 2.4	0.48 ± 0.04	0.27 ± 0.02
Gabbro	3	157 ± 3	58.1 ± 2.1	0.30 ± 0.03	0.25 ± 0.01
Charnockite	3	147 ± 3	59.5 ± 1.9	0.63 ± 0.04	0.17 ± 0.01

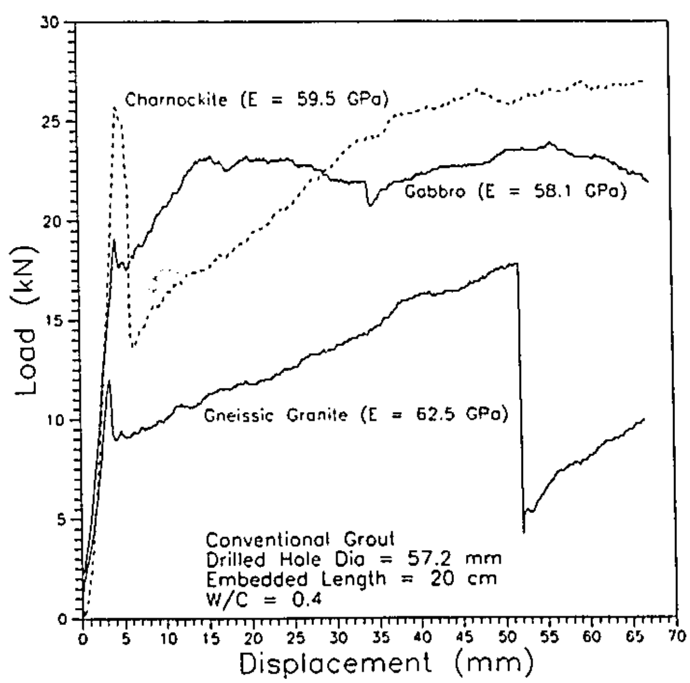


Figure 4.12: The effect of mechanical properties of rocks on cable pull out capacity (conventional grout)

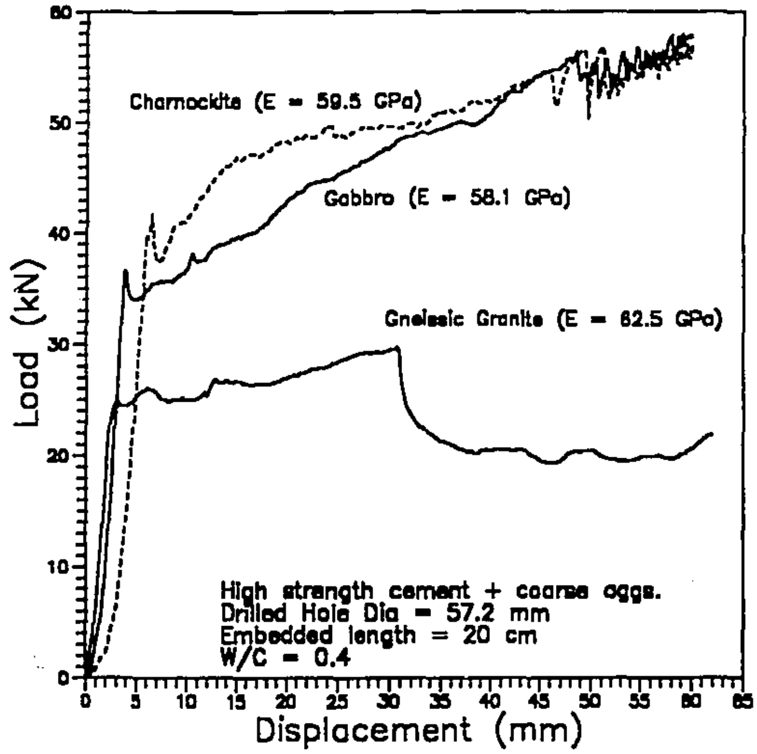


Figure 4.13: The effect of mechanical properties of rocks on cable pull out capacity (HSC + CA)

The following conclusions can be made:

1. All the rocks were from the category of hard rock. It was thought before commencing the experiments that the pull out capacity would be more or less the same. Definite differences were, however, observed, as shown in Figures 4.12 and 4.13.
2. These results seem to prove that the performance of fully grouted cables is sensitive to the mechanical properties (i.e. strain and Poisson's ratio) of rocks.
3. Gneissic Granite, which has the highest modulus of elasticity of the three rock types, found lowest in resisting the pull out force. This inferior behaviour could be attributed to relatively high Poisson's ratio and axial strain. Which further concludes that those

rocks which do not exert enough radial confinement on the grout column, cable pull out capacity gets reduced.

4. High strength grout increases the cable pull out resistance approximately 40 % more than the conventional grout.
5. The pull out curve of conventional grout drops significantly after passing the peak value. Whilst in HSCG, the curve drops slightly and then starts regaining its value. The ductility of system in all rocks with both grouts, however, remain identical.
6. No cracks were observed in the rocks around the cable after the pull out tests.

4.4.4 Rate of Displacement

This parameter was investigated by varying rate of displacement in laboratory experiments. The fast test rate of 1 cm/sec took 10 seconds to complete while the slowest test rate of 0.1 mm/min took almost 16 hours. Figure 4.14 shows that pull out resistance is independent of the selected rate of loading. Same graph shows that the fully grouted cables have a capacity to absorb the impact load. This is attributed to the gradual slip of cable in the post-elastic region. A twisted flexible cable, unlike rockbolts, does not crush the grout during the pull out process. The extension of this experiment, however, can not be applied in the field, where the embedment length is more than the critical length of the cable, for example, applications of 20 - 30 meter long fully grouted cables in open stopes. On the other hand, the advantage of such shock absorbing support is seen in the reinforcement of rocks immediately around an opening. During rockburst event, the explosive spalling of rocks endanger human lives and equipment. The size of these spalling rock blocks is, most often, less than the critical length of cable. Therefore, rock blocks reinforced with cable, do not get detached completely from wall, rather rocks keep on hanging with stable rocks through cables. Few blocks may fall down, however, a warning in the shape of slipping of rocks prior to complete failure will reduce the intensity.

Table 4.9: Different rate of loading and their effect on pull out capacity

Rate of Displacement	Average Residual Load (kN)			
	Test Number			Mean
	1	2	3	
1 cm/sec	55 *	57*	*	56.0 ± 1.41
1 cm/min	65	67	62	61.2 ± 5.12
1 mm/min	65	68	65	66.0 ± 1.73
0.1 mm/min	62	64	60	62.0 ± 2.00

* Due to fast rate, complete curve was not obtained in all tests.

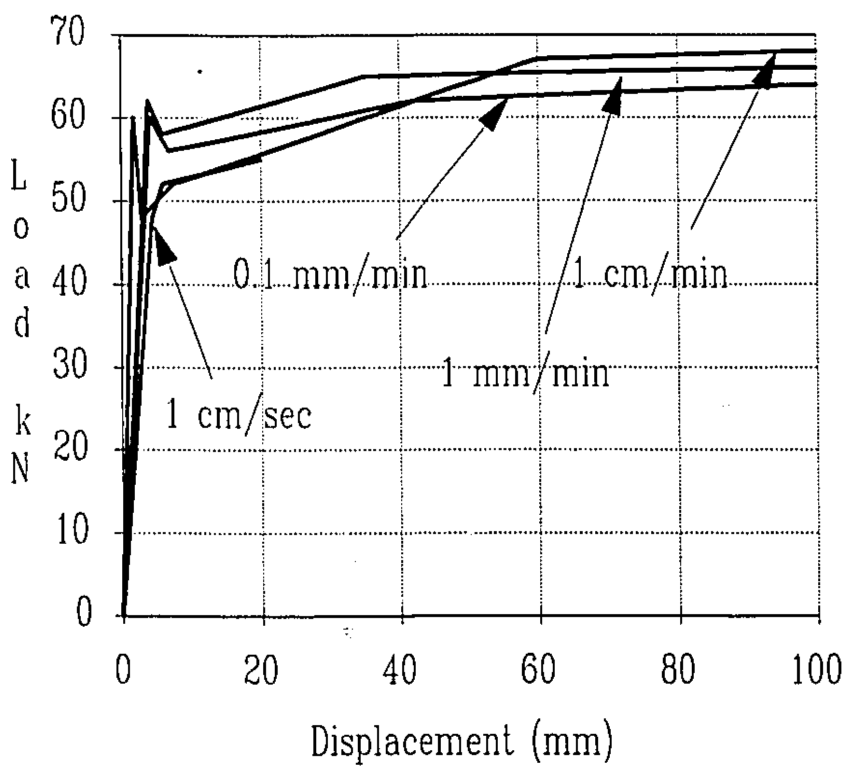


Figure 4.14: Effect of rate of displacement on pull-out resistance of cable

4.4.5 Water-Cement Ratio

The effect of water-cement ratio on the pull out capacity of fully grouted cables was evaluated by varying different embedment lengths against water-cement ratios. Schedule 40 steel pipes with inner diameter 57 mm were used to model host medium. Figure 4.15 shows the results of pull out tests for w/c = 0.3, 0.35 and 0.45. Best-fitted lines were drawn between the experimentally obtained points. A linear relationship was obtained for varying embedment lengths against selected water-cement ratios. The results of this parameter are equally important and applicable in the installation of cables in the field.

Table 4.10: Different water cement ratios and their effects

W/C	EL (mm)	Peak Load (kN)			
		Test Number			Mean
		1	2	3	
0.45	200	48.2	45.3	50.8	48.10±2.75
	400	81.0	77.8	83.1	80.63±2.67
	600	128.7	125.0	133.3	129.00±4.16
0.35	200	50.3	48.8	54.6	51.23±3.01
	400	100.0	97.2	104.6	100.60±3.74
	600	150.0	144.3	156.0	150.10±5.85
0.30	200	63.0	61.1	66.2	63.43±2.58
	400	121.5	118.4	126.8	122.23±4.25
	600	176.0	170.2	183.2	176.47±6.51

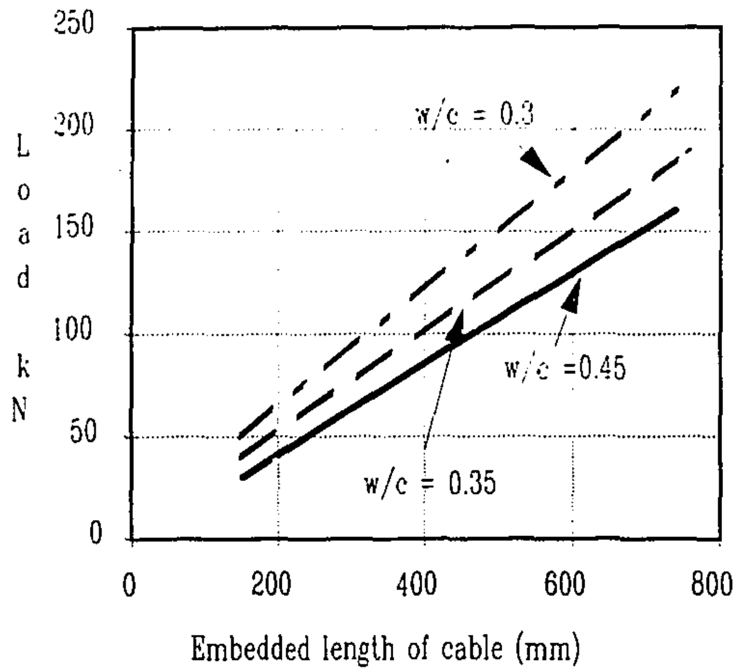


Figure 4.15: Effect of water cement ratio on the performance of cable bolt

4.4.6 Cable Embedment Length

The concept of cable bolting in rocks can also be defined as stitching of loose rocks with hard and intact rocks by means of long and flexible cables. This means the longer the embedment length the better the system should be. Figure 4.16 shows the load-displacement curves of embedment lengths ranging from 15 cm to 75 cm. The ultimate tensile strength of the cable is 270 kN. A maximum of 260 kN load was obtained in the laboratory. The extension of this parameter in the field is to design the fully grouted cable in a rock mass of having uneven spaced rock joints; see, for example, Figure 4.17. A rock block deeper than 80 cm and heavier than 260 kN can not be supported by a single cable. The other options should be used such as two cables per one drilled hole or to reduce the spacing between cables, etc. In case of excessive deformation of rock mass reinforced by cable bolting, the applied load on a single cable can not be determined precisely. Under such circumstances, the key factor is the joint spacing or in other words length of grouted cable should be taken as design parameter. Once again, if grouted length is more than 80 cm, and there is a potential of joint dilation, design should be modified to avoid failure. The maximum load carrying capacity of cable against embedment length is drawn in Figure 4.18. The following first order equation, applicable for an embedment length of 15 cm to 75 cm length, represents Figure 4.18, is shown below:

$$P = 3.82 * L_e - 40 \qquad 4.4$$

Where:

P = Load, kN

L_e = Embedment length, cm

Table 4.11: Embedment length as function of pull out capacity

EL (cm)	Average Residual Load (kN)			
	Test Number			Mean
	1	2	3	
15	20	18	23	20.33±2.52
20	37	35	40	37.33±2.52
30	52	47	55	51.33±4.04
40	100	95	102	99.00±3.61
50	153	148	156	152.33±4.04
60	199	193	203	198.33±5.03
75	240	230	*	235.00±7.07

* Cable failed at the chuck

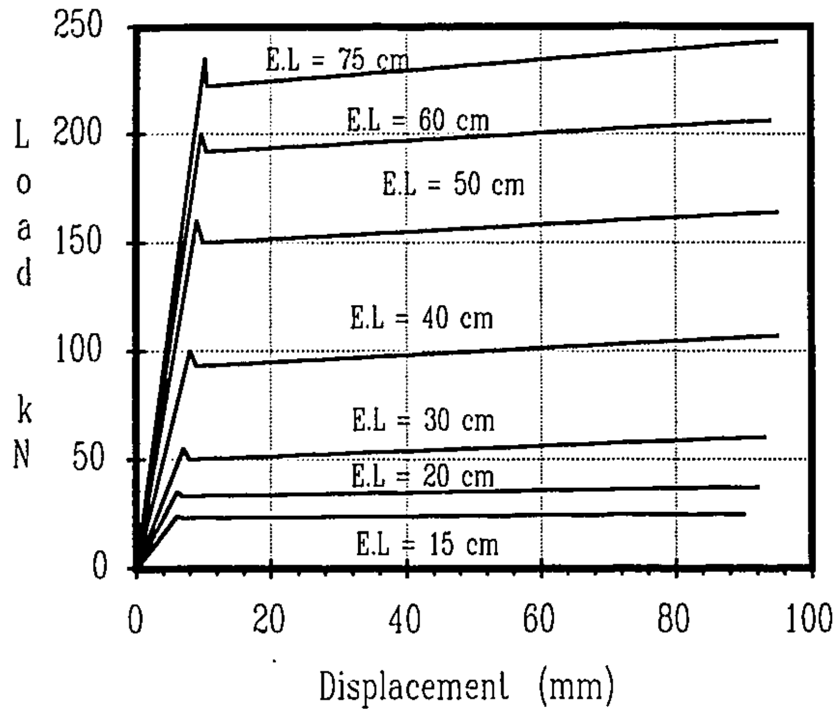


Figure 4.16: Effect of embedment length (E.L.) on cable-bolt capacity

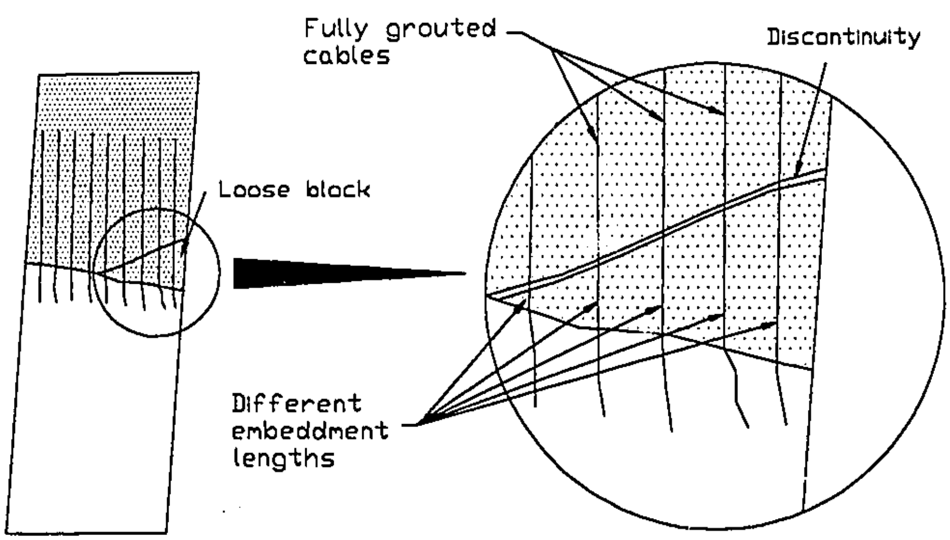


Figure 4.17: The concept of different embedment lengths of fully grouted cables in an underground excavation

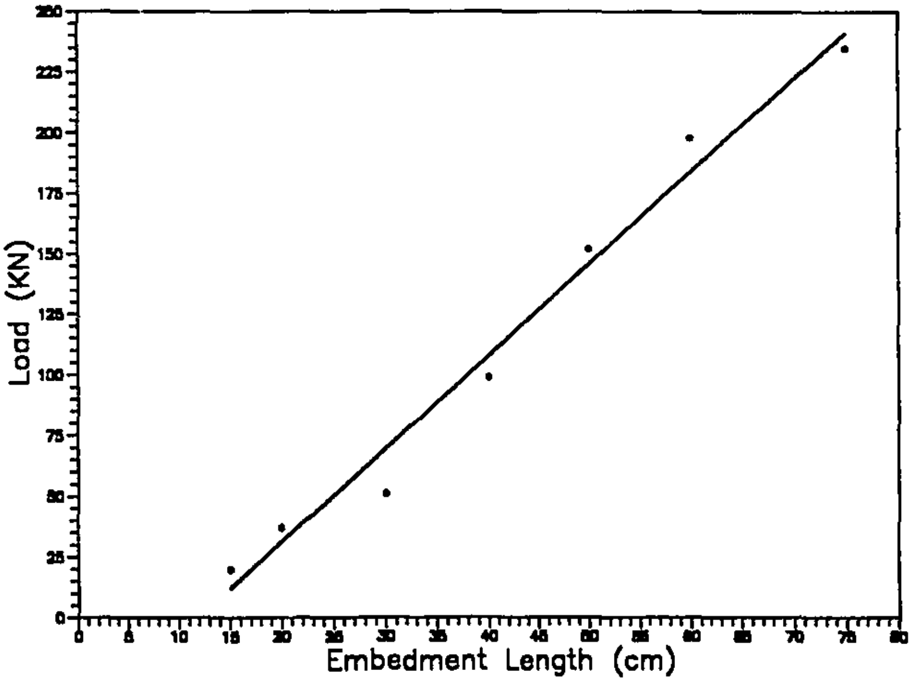


Figure 4.18: Maximum load carrying capacity versus embedment lengths of fully grouted cable

4.4.7 Curing Time

This factor determines when a grouted cable has attained its working capacity. From Figure 4.19, it can be seen that after 7 days, it is safe to operate under the grouted cables. The stiffness as well as strength of cable bolt system will however continue to grow until they reach their maximum values after 28 days. This parameter is equally applicable in the field provided drill holes are dry and free from running water. In the presence of excessive moisture or water, water resistant additives should be added to grout prior to its applications.

Table 4.12: Curing time as function of pull out capacity

Time (Days)	Average Residual Load (kN)			
	Test Number			Mean
	1	2	3	
1	60	52	65	59.0±6.56
3	78	71	89	79.3±9.07
7	162	155	174	163.7±9.61
28	240	230	*	235.0±7.07

* Cable failed at the chuck

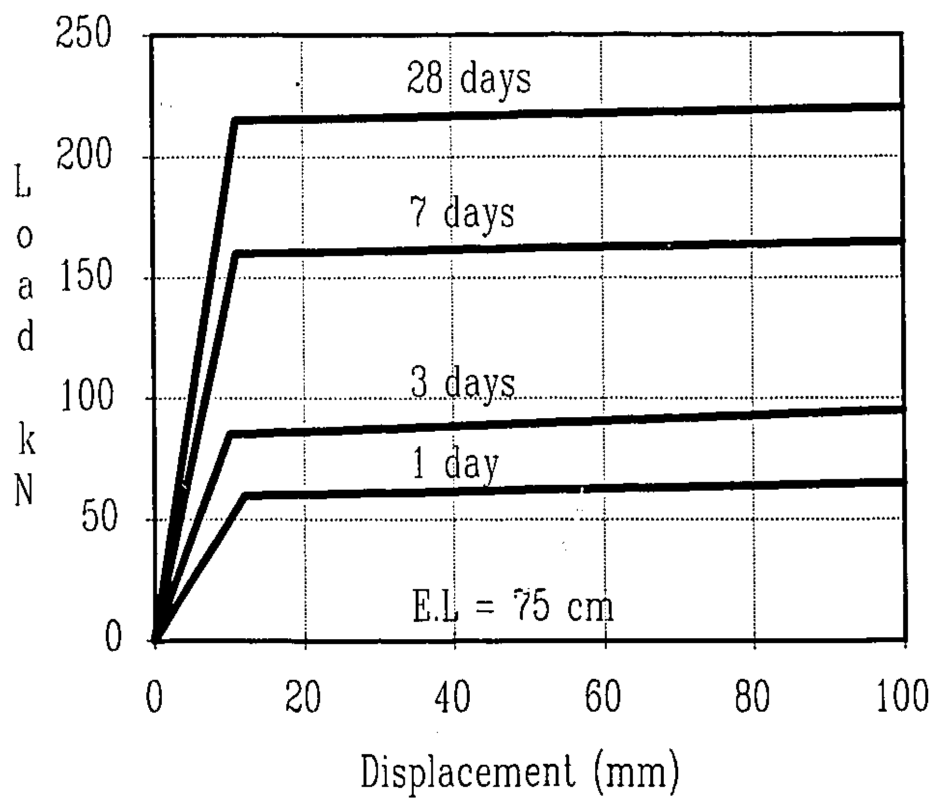


Figure 4.19: Load-displacement curves of various curing time of fully grouted cable

4.4.8 High Strength Cement Grout

High Strength Cement Grout (HSCG) is achieved by adding silica fumes in ordinary Portland cement. The ultimate uniaxial compressive strength of HSCG is approximately 100 MPa (E = approximately 45 GPa) and 50 MPa plus is obtained in 3 days. In this study, HSCG was used as a grout material with schedule 40 steel pipe as host rock. A water-cement ratio of 0.4 was selected to prepare the grout. Water-reducer and super plasticizer were added in the grout to increase the workability. Figure 4.20 shows two important factors: (1) the pre-slip peak value improves by 45 % and (2) the post peak value keeps on increasing; this is desirable in actual cable bolting. The system ductility as 10 cm of displacement were also achieved for HSCG tests. These results can be extended in field applications of fully grouted cable with HSCG.

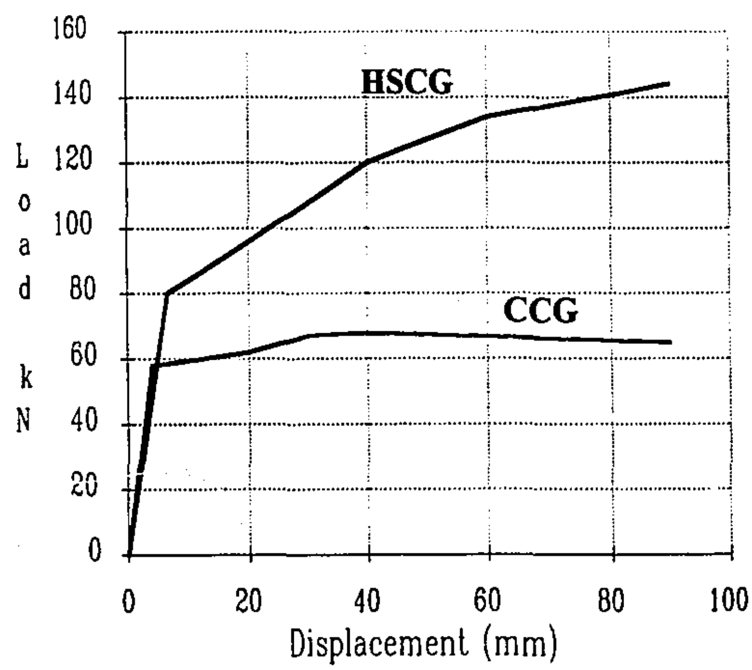


Figure 4.20: Comparison of Conventional Cement Grout (CC) and High Strength Cement Grout (HSCG)

Table 4.13: Comparison of conventional grout and high strength cement grout

Grout Type	Peak Load (kN)				Average Residual Load (kN)			
	Test Number			Mean	Test Number			Mean
	1	2	3		1	2	3	
Con.	59	56	62	59.0±3.0	67	65	70	67.3±2.52
HSC	80	77	83	69.5±11.81	120	117	135	124.0±9.64

4.4.9 Shotcrete Grout

The term 'shotcrete grout' means conventional cement plus coarse and fine aggregates. The presence of coarse aggregate in the grout can increase the frictional resistance between cable and grout during the process of slipping (Farah and Aref, 1986 and Rajaie, 1990). These researchers, however, added the fine aggregate (i.e. sand) in the shotcrete mix, which made the grouting process difficult and less efficient. In an effort to identify the best shotcrete mixture, the following mixes were used in this research:

1. Mix A; contains cement, fine aggregate (sand) and coarse aggregate (gravel or crushed aggregate (passing sieve size 9.50 mm (US 3/8) and retaining sieve size 2.36 mm (US 8)). The ratio of fine to coarse aggregate was used as 1:1 with water cement ratio as 0.5.
2. Mix B; contains only cement and coarse aggregate. A 0.4 water cement ratio was found to be appropriate for working.
3. Conventional grout with w/c = 0.4; for the purpose of comparison of results.

Figure 4.21 indicates that Mix B is superior to the others. Mix A was found troublesome because of mixing of sand; which requires more water and strict quality control. For Mix B, segregation of coarse aggregates should be avoided.

In the previous detail study of Mix A, all the important parameters were evaluated i.e. effect of water cement ratio, effect of embedment length (see Figure 4.22), etc. [Rajaie, 1990].

Table 4.14: Comparison of conventional grout and different mixes

Mix Type	Peak Load (kN)				Average Residual Load (kN)			
	Test Number			Mean	Test Number			Mean
	1	2	3		1	2	3	
Mix A	59	58	63	60.0±2.65	65	62	67	64.67±2.52
Mix B	76	74	81	77.0±3.61	80	81	83	81.33±1.43
Con.	53	50	58	53.67±4.04	67	65	70	67.33±2.52

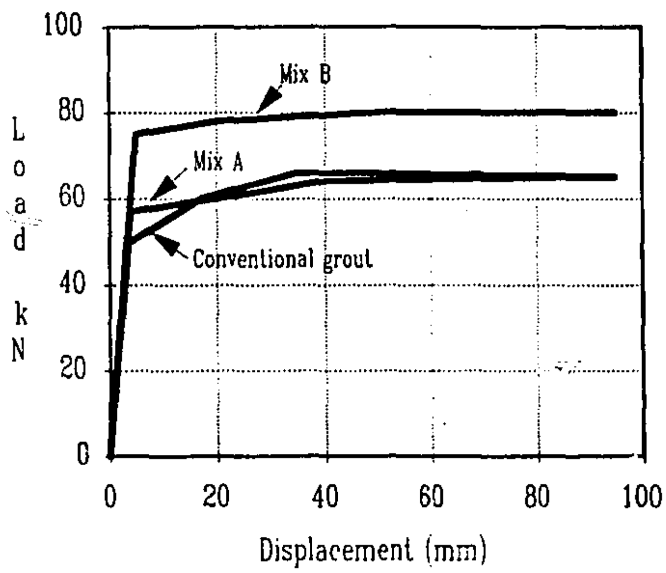


Figure 4.21: Conventional grout versus different concrete mixes

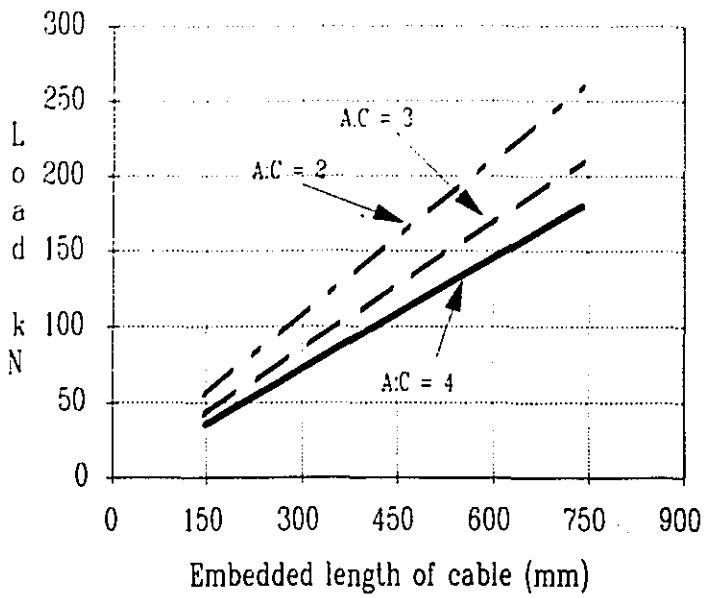


Figure 4.22: Effect of Mix A on embedment length of cable (A:C = Aggregate cement ratio)[Rajaie, 1990]

4.4.10 Cuttable Cable Bolts

Cuttable flexible fully grouted cable bolts are introduced in the mining with the introduction of continuous mining. Two types of cuttable cable-bolts are briefly discussed here (a greater detail is given is the following chapters). They are:

1. Arapree (ARAmid PREstressing Element) is made up of Twaron continuous aramid fibers pre-impregnated in epoxy resins. Arapree tendons are available in round and strip shapes with different surface profiles. Arapree has been used as tensile element to prestress concrete structures (Gerritse et al., 1987). The mechanical and physical properties of Arapree can be altered during its manufacturing process. The modulus of elasticity of Arapree is 125 GPa (steel = 200 GPa) and ultimate tensile strength based on net cross section of fiber content is approximately 2800 MPa (prestressing steel = 1700 MPa). A strip shape (b/d = 5/20 mm) Arapree flexible tendons were

used with conventional grout in the pull out tests. Steel pipes were used to model host rock. Figure 4.23 shows the results. A conventional steel cable test result is also presented for the purpose of comparison.

2. Polystal is a combination of continuous glass fibers reinforced and bonded with polyester resin. Four, 7-mm diameter strands in the birdcage formation were grouted in 30 cm long steel pipe. The ultimate load carrying capacity of four tendons is equivalent to single steel cable (2600 kN). A typical result of pull-test is shown in Figure 4.23 (Mah et al., 1991).

Table 4.15: The pull out capacity of cuttable tendons and conventional steel cable

Tendon Type	Peak Load (kN)				Average Residual Load (kN)			
	Test Number			Mean	Test Number			Mean
	1	2	3		1	2	3	
Steel	50	48	54	50.7±3.1	68	65	70	67.7±2.5
Arapree	59	55	63	59.0±4.0	24	24	28	25.3±2.3

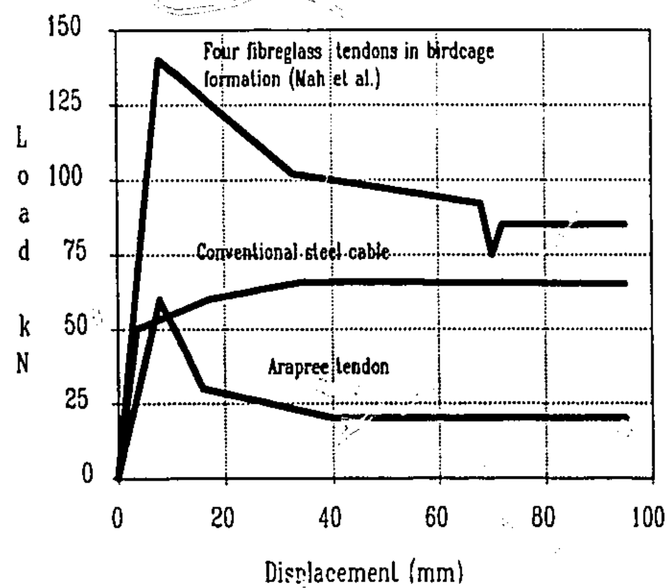


Figure 4.23: Comparison of different types of cuttable cable bolts

4.4.11 Shear Capacity of Fully Grouted Steel Cables

Various researchers have conducted shear tests previously, but most of them concentrated on steel bolts. Such as Bjurstrom (1973), Azuar et at. (1979), Hass (1981), and Barton and Bakhtar (1983) concluded that bolt shearing resistance capacity increases when the shearing load is applied at approximately 45° with the perpendicular axis of bolt. Ludvig (1983) performed shear tests in a large rig on Swellex, massive steel bolts, and massive and tube bolts of fiberglass. On the other hand, only Fines et al. (1977), Dight (1982) and Stillborg (1984) carried out shear tests on fully grouted steel cable. Fines et al. used steel casing as host medium and applied shear load perpendicular to the steel cables. Whereas Dight and Stillborg used rocks or concrete blocks to model host rocks. Both researches found that fully grouted cable capacity, like bolts, increases when the load is applied at 45° with cable axis.

The behaviour of cable as a shear resistant in a discontinuity is modelled by varying discontinuity spacing. This gives a combination of shear and tensile loading (see Figure 4.24). Steel cables were fully grouted in two 305 mm long schedule 40 steel pipe with a spacing between the steel pipes (see Figure 4.25). Conventional cement grout with water/cement ratio of 0.4 was used to prepare the samples. After 28 days of curing, samples were inserted and fixed in the shear testing frame and load was applied at 90° orientation to the joint. The rate of loading was kept constant at 1 mm/min, (displacement control).

A total of 18 shear tests was conducted. The effect of joint spacing on the performance of fully grouted cable to resist the shearing stress was the objective of these tests. The following were the joint spacing considered for the experiments:

- Joint spacing; 10 mm, 20 mm, 40 mm, 50 mm, 80 mm and 100 mm.

The test results are graphically presented in Figure 4.26. The average values of test sample, corresponding to dashed line drawn in the figure, are shown in Table 4.16. The following comments can be made from this figure:

1. The purpose of experiments was not to find the ultimate shear resistance capacity of fully grouted steel cables, but to see the combined shear and tensile mode of displacement and failure of shear reinforced elements. Therefore, experiments were stopped after a shear displacement of 50 mm.
2. From the Figure 4.26, it is obvious that the shear resistance capacity of cable reduces with the increase in joint spacing.
3. Cable, being flexible, was not found very effective in preventing the shear displacement of discontinuities. Cable started to bend after a shear displacement of only 1 or 2 mm. Cable, however, sustained effectively the shear load. This means that in practical situations, cable would sustain the shear load but allow the displacement of rocks. The rigidity of joints or discontinuities would not be maintained with the application of cables. Other types of rigid tendons such as steel rock bolts would seem to be more effective for such applications.
4. In all shear tests, the grout failed first and then the cable started to bend. Closer inspection of the test samples, after the tests were completed, revealed that in no test did any of the cable wire break.

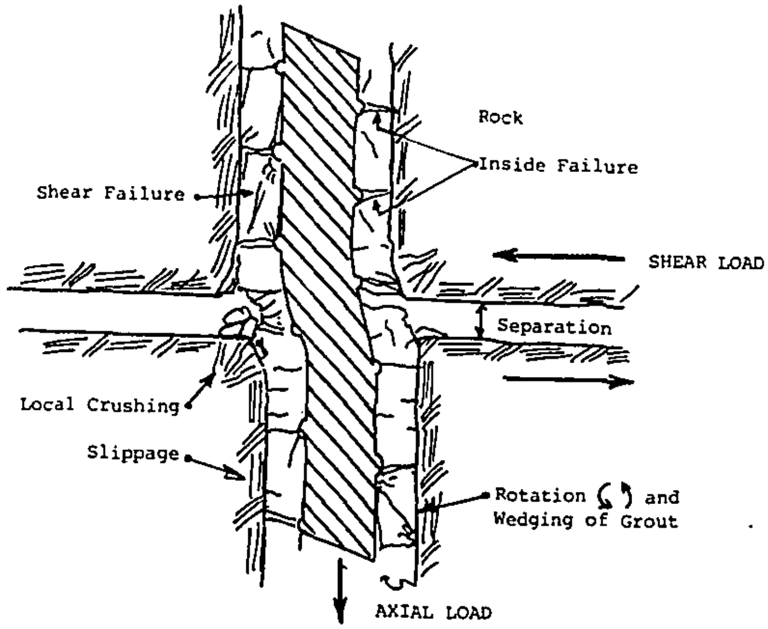
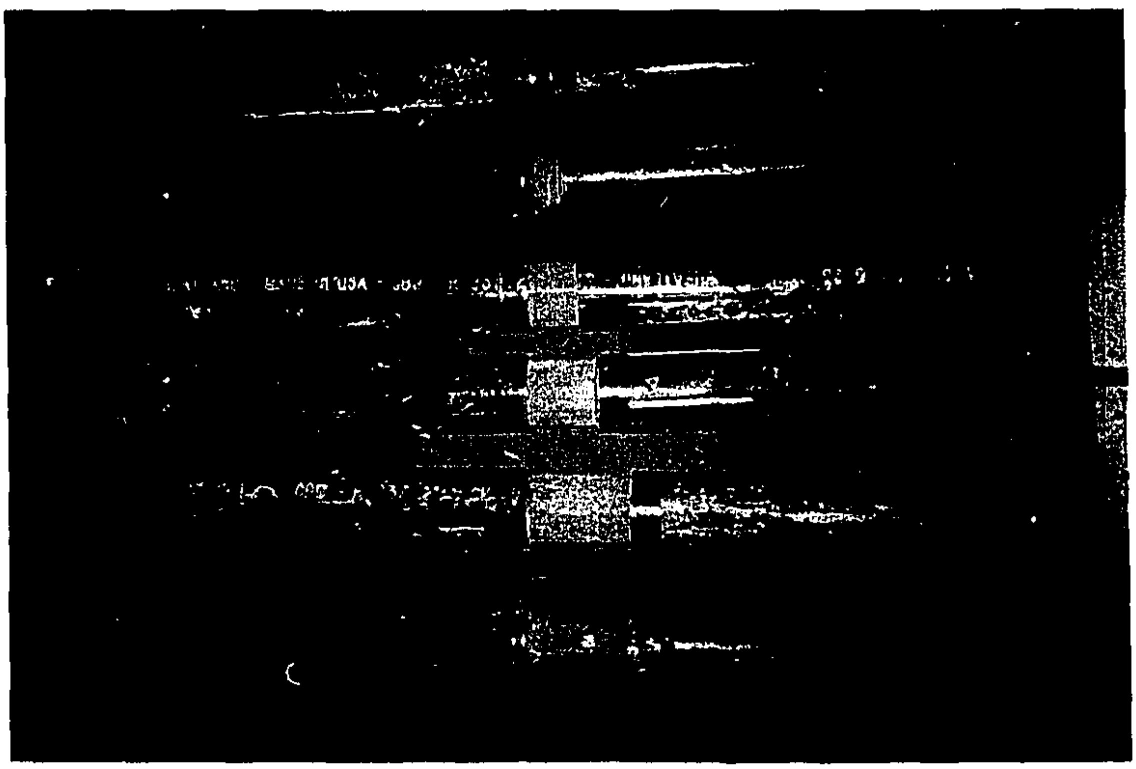


Figure 4.24: Concept of combined shear and tensile loading on a bolt functioning as shear resistant



Picture 4.25: Various joint spacing of the fully grouted steel cable samples, prepared for shear testing

Table 4.16: The effect of joint spacing on the shear resisting capacity of fully grouted steel cable

Joint Spacing (mm)	Average Shear Load (kN)*			
	Test Number			Mean
	1	2	3	
10	85	88	93	88.67 ± 4.04
20	73	74	77	74.67 ± 2.08
40	42	48	50	46.67 ± 4.16
50	15	17	18	16.67 ± 1.53
80	14	15	18	15.67 ± 2.08
100	5	6	8	6.33 ± 1.53

* Data representing to the dashed line, see Figure 4.26

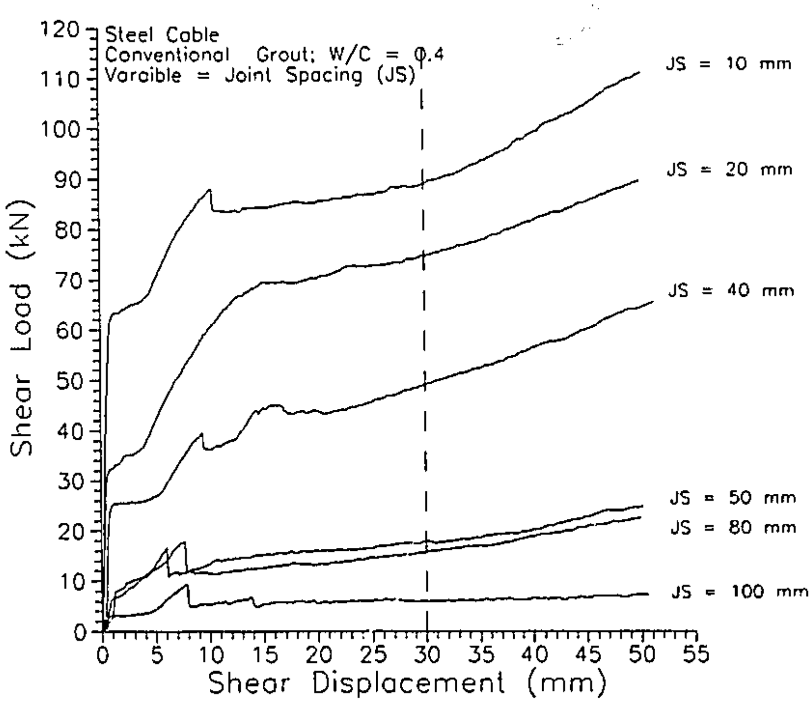


Figure 4.26: The effect of joint spacing on the shear resistance capacity of fully grouted steel cable

4.5 Discussion

The laboratory investigation in this chapter considers a number of parameters, not previously considered, relating the performance of fully grouted steel cables (strands). The basic purpose was to provide insight into the interaction between cable-grout and grout-rock. All the experiments were conducted in the laboratory, however, their extension in filed is also possible.

Fully grouted 7-wire flexible steel cables were used in all the experiments, except that, two composite flexible tendons were also introduced and tested for the purpose of comparison. Various types of bonding materials, such as conventional cement grout, high strength cement grout and shotcrete grout were used as bonding agent between cable and host medium in the experimental program.

The effect of hole diameter of the loading plate was also evaluated, however, the implications of this test is limited to the laboratory only. It has been shown that the effect of strength and stiffness of the grout and host medium is significantly large. Radial confinement pressure exerted by host rock mass was also modelled in laboratory. The interaction between the mechanical properties of the grout and host medium along with the confining pressure seemed to effect the performance of the fully grouted cables.

Another series of experiments on the rate of displacement of fully grouted cables were undertaken. Results show that, within the experimental range of loading rates, the performance of cable is independent of such rates. This indicates that cable support systems are effective under dynamic loading such as rockbursting and blasting.

A linear relation was observed between cable embedment length and pull out capacity. The critical length of a cable bolt was found approximately 80 cm. The gradual increase in bonding strength of grout with curing time was also been evaluated by pull out tests.

A comparison of high strength cement grout and conventional grout was also made, which showed that the former has 45 % more bonding strength than the latter. The ductility of a system, however, remains unaffected. Different mixes of shotcrete were also tested. A mix containing cement and coarse aggregate was found superior to conventional grout.

A detailed experimental description of cuttable (composite) bolts is given in the forthcoming chapter, however, for the purpose of comparison, the pull out capacity of two types of cuttable bolts, Arapree and Polystal, were compared with conventional steel cable. Results indicate that the performance in the elastic range (pre-slip) is identical, however, in the post elastic range, conventional cable gives more residual load carrying capacity.

The shear resisting capacity of fully grouted cables reduces with the increase in joint spacing. Cable, being flexible, was not found to be very effective in preventing shear displacement. Cables started to bend after a shear displacement of a few millimeters, they, however, sustained effectively the shear load.

Chapter 5

Introduction to Composite Materials

5.1 Introduction

The technological advantages of composite materials were realized by scientists and engineers even before 1940. However, it was the demand generated by World War II to produce structural elements superior in mechanical and physical properties than conventional materials that led to their development. The success can be viewed in a large quantity of composite materials utilized in various fields. For example, in 1979, the industry produced some 8 billion pounds of various types of composites including 2 billion pounds of reinforced fiber glass products, 2 billion pounds of asbestos reinforced products and 4 billion pounds of products using cellulosic fiber, cotton, polyamide (nylon), and sisal. The total value of these composites to the industry was about 6 billion US dollars (Rosato, 1982).

The aerospace industry pioneered the use of composite materials as tensile, compressive and flexural load bearing elements and as non-structural parts such as seats, cabins, windows, doors, etc. Composites also became popular in other fields as soon as their prices dropped to competitive level of other conventional materials. The manufacturing of boats, cars, housings appliances, trays, storage containers are only a few examples which benefited from the advantages of composite materials.

There are two basic reasons for the widespread popularity of composites: (1) their high resistance to corrosion, and (2) their high strength-to-weight ratio. The second property makes them very economical for high fuel consumption industry. This chapter aims to provide basic information on composite materials their structure, types, advantages and applications in industry particulary in mining.

5.2 Definition

According to Rosato (1982) Composites can be defined as "*the combination of two or more materials, present as separate phases and combined to form structures that take advantages of certain desirable properties of each components. The constituent materials can be organic, inorganic or metallic in the shape of particles, rods, fibers, plates, foam, etc.*" More specifically, suitable fibers are combined together and held in any desirable shape by binder materials, known as the matrix. Fibers are the load bearing component and the matrix materials are responsible for the dimensional stability of the product and the protection of the fibers.

Fiber-reinforced composites are of two types - continuous and discontinuous or short-fiber composites, i.e. the properties of composites change with fiber length. Continuous fiber composites are those where there is no change in the elastic modulus of the composite caused by any increase in fiber length. The application of short fibers is in plates, shells, beams, column, etc., where structural elements should sustain both compression and tension forces. The cross-sectional area of short-fiber composites are more than their counterpart. Continuous fiber composites are primarily manufactured to replace heavy, tensile bearing structural elements. Such geometry makes composites weak under compression load. The other advantage of combing continuous fiber is that it makes the end-product a redundant material i.e. failure starts with the breaking of a few fibers and simultaneously redistributes the load onto other fibers; this thus prevents a catastrophic failure.

In the beginning, composites were not very attractive due to their very high cost and poor mechanical properties. Since 1970, however, improved raw materials as well as low cost, and efficient manufacturing have resulted in increased production of composites with much improved mechanical properties. Figures 5.1 and 5.2 give an idea of the changes in composites production and cost over the last two decades.

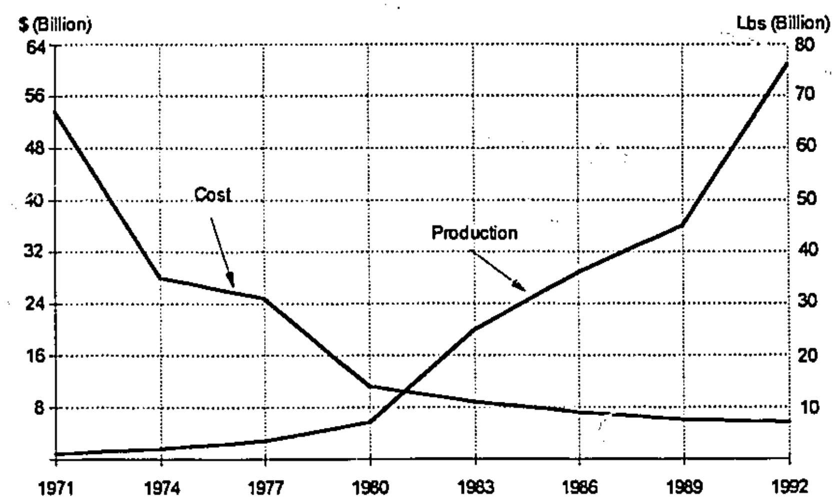


Figure 5.1: Cost and production of composite materials since 1971 (Technical brochure, BFGoodrich, 1986)

5.3 Raw Materials

Different types of fibers and matrices are commonly used in the manufacturing of end-products of composite materials. The mechanical and physical properties of end-products are heavily dependent on properties of raw materials. The following is a brief description of all those raw materials which are most often used in mining related products.

1. Glass Fiber
2. Carbon or Graphite Fibers
3. Aramid Fibers (Kevlar and Twaron)

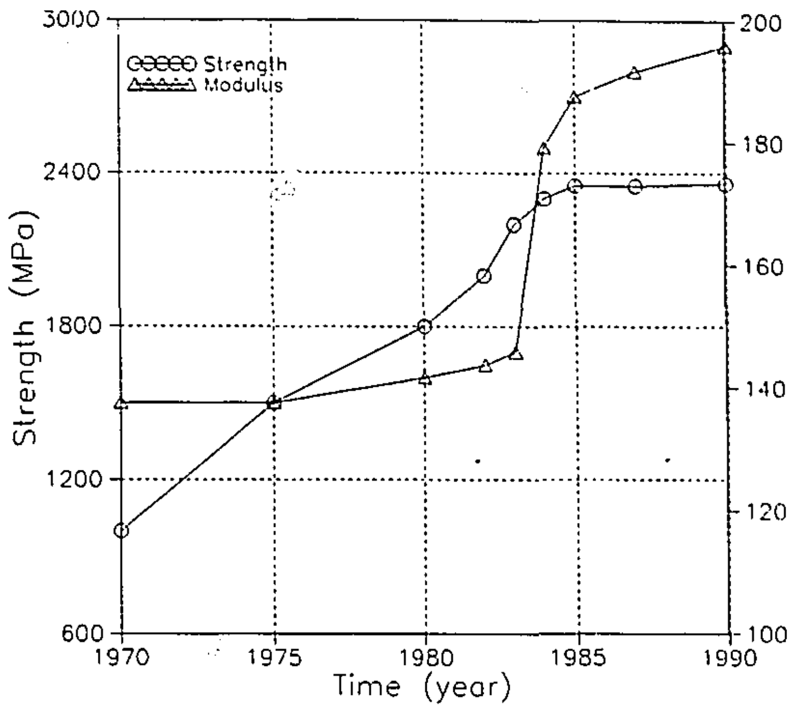


Figure 5.2: Improvement in the strength and modulus of composites since 1970 (Technical brochure, BFGoodrich, 1986)

5.3.1 Glass Fiber

High strength glass fiber, developed in 1960's, was a key technological development. The uses of glass fiber reinforced composite range from non-structural, low performance applications, such as panels in aircraft and appliances, to high performance applications, such as rocket motor cases and pressure vessels. Competitive prices, availability, ease of processing, and high strength are the reasons for widespread glass fiber use. Its performance, however, was still poor against aggressive environment and long-term loading at high stress levels. There are several types of glass fibers available for different applications. The most of the engineering properties of these glass fibers are given in Table 5.1 (Knox, 1982). The following is the list of various grades of glass fiber:

- A-glass
- C-glass
- E-glass
- S-glass
- M-glass
- D-glass
- L-glass

High-alkali glass or Soda-lime silica (A-glass) is the most common composition of fiberglass products. It is generally used where chemical resistance is desired such as manufacturing of containers and plate glass. A low-alkali composition or alumino borosilicate (E-glass) exhibits excellent electrical insulation properties. Presently, E-glass constitutes the majority of textile fiberglass production. For extremely good chemical resistance, sodium borosilicate (C-glass) fibers are used to meet the desired properties.

For application demanding high tensile strength (such as aircraft structural elements), magnesium aluminosilicate (S-glass) fibers have been used successfully. The tensile strength (in single fiber form) of S-glass is approximately 40 % higher than E-glass. Moreover, S-glass retains significantly higher tensile strength than E-glass at elevated temperature.

The M-glass composition was produced to obtain products of high modulus of elasticity ($\approx$ 112 GPa). The presence of beryllia (beryllium oxide) in the glass prevented its commercialization. The D-glass composition or low-dielectric glass has dielectric constant of 3.8 where E-glass has 5.9. This makes D-glass a suitable choice for high performance electronic applications. The L-glass (lead glass) composition has advantages of radiation protection and it is used in such applications.

The widespread application of fiberglass in multiple industries are due to the following of its inherent properties:

1. High tensile strength.
2. High resistance to moisture (swelling, stretching or disintegration) (Knox, 1982).
3. Strong resistance to heat, fire and high temperatures.
4. High resistance to chemical degradation.
5. Fiberglass is non-conductive with high dielectric strength and low dielectric constant.

Table 5.1: Properties of different grades of Fiberglass (Knox, 1982)

Properties	Grade of Glass			
	A	C	E	S
Specific gravity	2.5	2.49	2.54	2.48
Tensile strength (MPa)				
At 22 °C	3033	3033	3448	4585
At 371 °C	-	-	2620	3758
At 538 °C	-	-	1724	2413
Tensile modulus of elasticity at 22 °C (GPa)	-	69	72.4	85.5
Strain at failure, %	-	4.8	4.8	5.7
Elastic recovery, %	-	100	100	100
Coefficient of thermal linear expansion, m/m/°C	8.6	7.2	5.0	5.6
Specific heat at 22 °C	-	0.212	0.197	0.176
Softening point, °C	727	749	841	-
Dielectric constant at 22 °C				
At 60 Hz	-	-	5.9 - 6.4	5.0 - 5.4
At 10^6 Hz	-6.9	7.0	6.3	5.1

5.3.2 Carbon or Graphite Fibers

According to Riggs et al. (1982), *"carbon fibers are by far the predominant high-strength, high modulus reinforcing agent currently used in the fabrication of high performance resin-matrix composites"*. Graphite (and carbon) fibers consist of perfect crystal fibers, therefore, their products are highly anistropic in nature. The difference between graphite and carbon fibers is that former is heat-treated at temperature in excess of 1700 °C and posses high degree of preferred orientation, whereas, latter is processed at temperature lower than 1700 °C and has lower degree of preferred orientation.

Commercially available carbon fibers have a wide range of tensile modulus, ranging from 270 GPa to 517 GPa. In general, the low-modulus fibers have lower specific gravity, lower cost, higher tensile strengths, and higher tensile strains at failure then high-modulus fibers. The advantages of carbon fibers are: their exceptionally high tensile strength-to-weight ratios, tensile modulus-to-weight ratios, very low coefficient of linear thermal expansion, and high fatigue strengths. The disadvantages are their very high cost, low impact resistance and high electric conductivity. Their high cost has limited them to aerospace industry where weight savings is considered more critical than cost. The product of carbon/graphite fibers have not been commercially available in civil or mining engineering. However, the Japanese are working presently to produce carbon prestressing tendons (Minosaku, 1992).

5.3.3 Aramid Fibers

Unlike glass-fibers, aramid fibers (Kevlar and Twaron) are used only as continuous fibers, so their applications are limited to tires, hoses, conveyors, cables, and other fields where tensile load bearing is the main function. Due to its high cohesiveness, aramid can absorb greater energy than brittle fibers, this makes them suitable for lightweight armour. Aramid fibers are five times stronger than steel of the same weight and ten times stronger than aluminum. High strength, high modulus, and heat and corrosion resistance are fundamental properties of aramid fibers. These were commercially introduced as Kevlar by DuPont in 1960. Twaron, similar fiber to Kevlar, introduced by Akzo, a joint venture of The Netherlands and Germany (Schurhoff and Gerritse, 1986). Both fibers are developed and commercialized individually. Kevlar has not yet developed tensile bearing elements for mining applications, however, their high strength and low weight ropes are widely used in under sea applications. On the other hand, Arapree - an end-product of Twaron fibers, are finding wide range of application in prestressing concrete industry in place of steel strands.

The potential applications of aramid cable bolts and rockbolts in underground mining are considered to be significant. The end-products of aramid are manufactured through a pultrusion process.

Kevlar

Kevlars are basically aramid fibres (generic name for aromatic polyamide fibres). All types of Kevlar (Kevlar 29, Kevlar 49, Kevlar 149) are made by solution-polycondensation of diamines and diacid halides at low temperatures. The polymers are spun from strong acid solution by a dry-jet wet spinning process. Principally, the polymers are made by rapidly adding a diacid chloride to a cool amine solution, with stirring. Kevlar properties can be altered either by varying solvent additives or by varying the spinning conditions or by using post-spinning heat treatment.

Aramid fibres are inherently resistant to flame and higher temperatures, as well as to organic solvents, fuels and lubricants. The thermal properties of Kevlar are presented in Table 5.2. It is obvious that tensile strength decreases as thermal conductivity increases with very high temperature. Kevlar 29 fibre is chemically quite stable and its resistance to natural chemicals is generally very high; whereas Kevlar 49 is susceptible to attack by acids and bases, especially strong acids.

The mechanical properties of the Kevlar fibres differ from the other organic fibres. Kevlar fibres have very high tensile strength, initial modulus, and low elongation with a linear stress-strain relation. Table 5.3 summarizes the mechanical properties of Kevlar.

By increasing the strain rate from 0.167 to 8000 % per second, the tensile strength of the dry Kevlar fibre drops an average of 14 %; the rupture elongation remain virtually unchanged (Chiao and Chiao, 1982). This property would yield support behavior useful for mine stability. The effect of temperature on the tensile properties of the dry yarn fibres is not great. Kevlar cable-bolts in hardrock mine structures would give a high residual stress because Kevlar with very high strength and low (or high) modulus can also be achieved by altering the manufacturing process.

After being bent for a short time at 21°C and then released, Kevlar fibre exhibits good recovery. However, the recovery decreases when the specimens are bent for longer times and/or at higher temperature (Chiao and Chiao, 1982). Bending properties would have little or no effect on the load bearing mechanism of Kevlar cable-bolts when used to sustain tensile stresses in mining applications.

Table 5.2: Thermal Properties of Kevlar 49 Yarn and Roving [Chiao and Chiao, 1982]

Property	Value
Decomposition temperature, °C	500
Tensile Strength, MPa	3620
At room temperature for 16 months	No strength loss
At 50 °C in air for 2 months	No strength loss
At 100 °C in air	3170
At 200 °C in air	2720
Tensile modulus, GPa	117
At room temperature for 16 months	No modulus loss
At 50 °C in air for 2 months	No modulus loss
At 100 °C in air	113.8
At 200 °C in air	110.3
Shrinkage, %/°C	4×10^{-4}
Thermal coefficient of expension, 10^{-6} cm/cm °C	
Longitudinal, 0 - 100 °C	-2
Radial, 0 - 100 °C	+59
Specific heat at room temperature, J/g °C	1.24
Thermal conductivity at room temperature, J.cm/sec.m^2.°C	
Heat flow perpendicular to fibers	4.110
Heat flow parallel to fibers	4.816
Heat of Combustion, kJ/g	34.8

Table 5.3: Mechanical Properties of Kevlar Fibres [Chiao and Chiao, 1982]

Property	Kevlar 29	Kevlar 49	Kevlar 149
Tensile Strength, MPa	2930	3620	3447
Young's modulus, GPa	62	117	172
Strain at failure, %	4.4	2.9	1.9

Twaron

Twaron is an aramid fiber produced by Enka BV and N.V. Noordikiijke, The Netherlands. The molecular chain of Twaron aramid fiber are aligned and rigidly linked by bridges of hydrogen. This high paracrystalline order is responsible for resistance against high temperatures. Like Kevlar, Twaron fibers have a very high tensile strength and modulus. There are two types of Twaron aramids: Standard and High Modulus (HM). The mechanical and physical properties of Twaron fibers are given in Table 5.4 (Gerritise et al., 1987).

The performance of Twaron fibers under constant load can be seen in Figure 5.3, which indicates that the creep rate is nearly constant per decade. The creep rate is about 0.25 mm/m per decade for high modulus Twaron fibers stressed at about 50 % of their ultimate tensile strength at 20 °C. The properties of Twaron fibers do not significantly change at elevated temperature or under prolonged heating; see Figure 5.4. The strength, however, at low to very low temperature increases but without becoming brittle. The coefficient of thermal expansion of the fibers is slightly negative (Gerritse and Werner, 1988).

There are other fibers such as boron, silicon carbide, polyethylene, etc. but they do not meet the requirements of mining industry.

Table 5.4: Mechanical and physical properties of Twaron HM fiber (Schurhoff and Gerritse, 1986)

Properties	Units	Short term	Long term
Density	Kg/m^3	1450	1450
Tensile strength	MPa	3150	2000
Characteristic strength	MPa	2400	1400
Young's modulus	GPa	125	90
Elongation at break	%	2.0	--
Thermal expansion	10^{-6} /K	-2	-2
Max. working temperature	°C	250	250
Glass transition temperature	°C	670	670

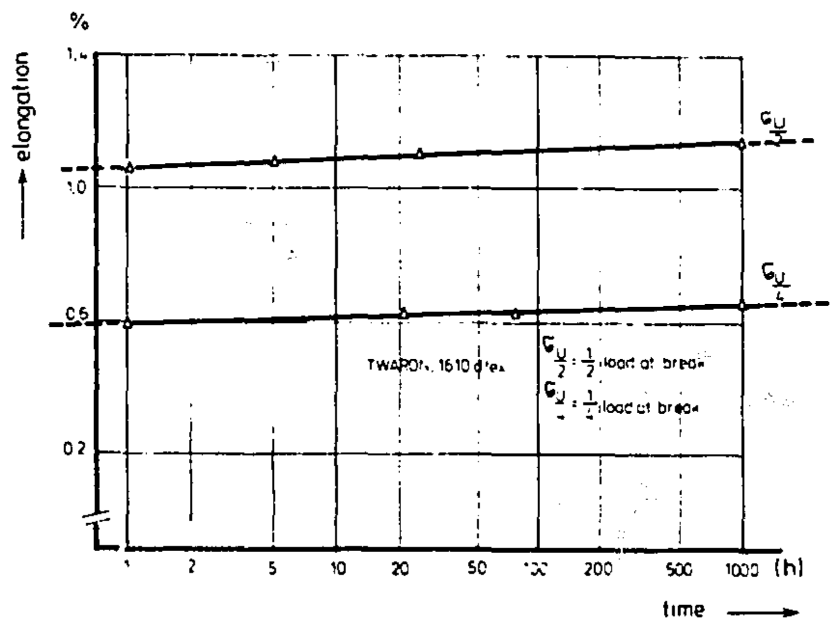


Figure 5.3: Creep of Twaron HM fibres (Gerritse and Werner, 1988)

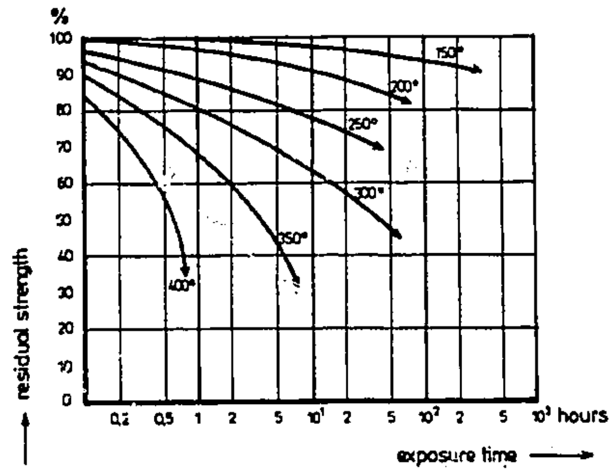


Figure 5.4: Influence of prolonged heating on Twaron HM fiber (Gerritse and Werner, 1988)

5.3.4 Matrices

Fibers and matrices are the main constituents of composite products. Fibers are designed to take applied stresses and matrix to bond the fibers together. The role of the matrix in the fiber-reinforced composite is the following:

- To transfer stresses between the fibers.
- To protect fibers against damaging environment.
- To make composite products dimensionally stable.
- To protect the surface of fibers from mechanical abrasion.

Matrix plays only a minor role in the tensile load carrying capacity of composite, however, in-plane and inter laminar shear strength of fibers depend heavily on the quality of the matrix. Damage tolerance of composite structures also depend heavily on interaction between fibers and matrix. Moreover, the processability and stability of composite materials depends on the matrix physical characteristics, such as viscosity, melting point, and curing temperature. Long term performance of composites are equally shared by fibers and matrix.

There are two categories of bonding matrices; thermoplastic and thermoset. Latter polymers (also known as resins) have been widely used as matrix material in the fiber reinforced industry. Fibers are either pulled through or immersed in low viscosity thermoset polymers before the polymerization reaction begins. A high temperature and pressure is usually not required because of low viscosity. The advantages of thermoset polymers are their high thermal stability and high chemical resistivity. They also exhibit much less creep and stress relaxation than thermoplastic polymers. Their limited storage life at room temperature, long fabrication in the mold and low strain at failure are some of the disadvantages. Their low strain contributes to the low impact strengths of final products. Thermoset resins are brittle to some extent and non-recyclable because once cured, they cannot be reused.

Thermoplastic polymers have higher impact and fracture resistance than their counterparts. These two properties make their end-products excellent damage tolerant. Due to their high strain to failure, a high resistance to microcracking in the composite laminate is obtained. Thermoplastic resins are capable of being modified by variation of temperature, and therefore, are more environmental friendly. Disadvantages are their high melt or solution viscosity, which make difficult incorporation of continuous fibers into the moulds. Their low creep resistance and thermal stability has discouraged their use in structural applications.

5.4 Tensile Failure of composite Elements

Failure of a tensile bearing elements under excessive uniaxial tensile loading can differ widely depending on the properties of constituent materials. Figure 5.5 shows three types of commonly observed failures (Tsai, 1988). In an ideal situation, all the fibers should take equal stress, and when applied stress is more than the strength of fibers, then they should fail suddenly, as shown in Figure 5.5 (a). However, due to presence of defects and weakness, fibers do not carry equal stress. Thus, due to breaking of few fibers, crack starts growing into the matrix. The path of crack depends mainly on matrix and interface properties. If bonding is strong, the crack tends to grow into the matrix, exhibiting a fairly smooth surface across the section with some of the fibers pulled out; as shown in Figure 5.5 (b). If bonding is not strong, irregular failure will be obtained due to the growth of cracks into fiber-matrix interface. Thus, few fibers will be pulled out from different locations and cracks into matrix will also be visible. Such a failure is shown in Figure 5.5 (c).

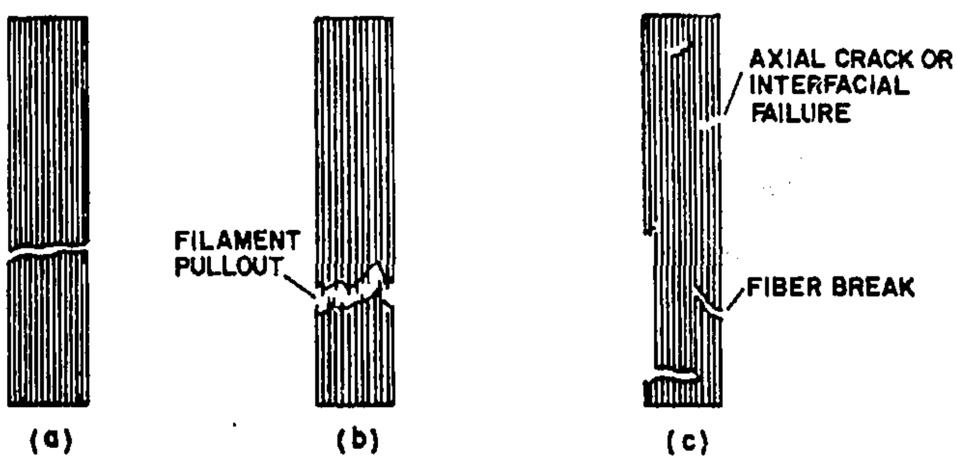


Figure 5.5: Longitudinal tensile failure modes (a) brittle, (b) brittle with filament pull out, (c) irregular (Tsai, 1988)

5.5 Manufacturing Processes

There are three methods of producing composite end-products: lay-up, pultrusion and filament winding.

Hand lay-up or spray-up is the easiest method; chopped or continuous fibers are sprayed in different layers on the mold surface. These layers are bonded together by matrix materials. Numerous reinforcement additives and fillers can be combined with the resin to customize the mixture to meet specific requirements.

Pultrusion or a Continuous Manufacturing Process is a continuous feed of fibers, in selected directions and then impregnated with resin and pulled through a heated die to give the shape of the final section (e.g., tubes or I-beams). Partial or complete curing occurs during passage through the die. Braiding is further developed in the pultrusion process. In braiding, continuous fibers in the shape of threads are mounted (as spindles) on the "carrier". (Carrier refers to the piece of machine that holds the spool of raw material to be braided). Continuous threads are pulled through the pre-sized, pre-arranged orifice and carrier spines at the same time. In this process flexible tubular tendons are impregnated into epoxy, which on curing gives a solid tendon. Stiffness of the tubular structure depends on the matrix, while mechanical properties depend on the orientation of fibers. Most of the mining compatible tendons are manufactured by pultrusion process.

Filament Winding Continuous reinforcement in the form of roving or mono-filaments are wound over a rotating mandrel. Specially designed machines, traversing at speeds synchronized with the mandrel rotation, control the winding angles and the placement of the reinforcement.

5.6 Effects of Environment on Composites

The following are the key concerns of composites due to different types of environmental effects. The following discussion is derived from Harris (1986), Whitney et al. (1980), Tsai (1988), and Staunton (1982). These environmental concerns are general in nature, while mining specific concerns are discussed in the sub-section 5.6.1.

- The effects of temperature, ultra-violet radiation and chemical environments including oxidizing atmospheres are most significant on the performance of composite materials. But, moisture with or without presence of chemicals, is likely to concern most of the designer. Most often, it is the matrix which gets effected due to severe environment, rather than the fibers.
- At elevated temperatures, creep may occur either in fiber or matrix. No creep occurs in glass, carbon, or boron fibers over the temperature range within which most resins remain undamaged.
- The time dependent behaviour of (long) continuously-fiber-reinforced composites are depend on extent to which the continuous fibers support the major part of the load. In discontinuous (short) fiber composites, however, failure starts due to degradation of matrix and fiber may remain unaffected and undamaged.
- Most forms of high energy radiation are damaging to polymers because of the relatively low energies required to cause chemical damage. The most obvious effects in long-chain polymers (i.e. Kevlar and Twaron) are the degradation associated with scission of the main chains which results in a reduction in the average molecular weight and additional cross-linking between chains resulting in network formulation.
- Outdoor exposure results in serious damage to the surface layers of laminates. The extent of damage varies merely colour fadding to completely removal of resin-rich surface layer or jet-coat and exposing the underlying fibers to mechanical damage and

further environmental attack. This kind of composite damage is likely to be the result of the combined effects of ultra-violet, regular wetting and drying, and everyday temperature cycling.

- In metal-matrix composites, an aqueous environment may cause severe corrosion. Since, these composites are not identified as useful alternative in mining, therefore, they are not added in the research program and not discussed in this section.
- Water diffuses easily through many of the thermoset and thermoplastic polymers that are commonly used as matrix materials. Water, usually, "wicked in" by capillary along the interface when ends of fibers, for example, cut edges of components, come in contact with excessive moisture (or water). Moisture, therefore has ready access to those parts of composite from where it drives the load bearing properties, to a greater or lesser extent, depending on the nature of the matrix polymer concerned, and in the long term this can result in deterioration of composite properties.
- Continuous (long) glass fiber composites are, in general, most sensitive to hydrothermal effects. Due to capillary action of exposed fiber ends, water, in an ambient temperature, gets absorbed by fibers and through fiber-matrix interface. This moisture over the time reduces the strength of composites.
- In most engineering usage, composite load bearing elements are designed based on the strength and stiffens of fibers. Bonding material i.e. matrix is considered only a bonus in the performance of mechanical properties of structural elements. Thus, some matrix softening due to moisture absorption (or moisture diffusion) can be accommodated in such applications without serious consequences. If considerable matrix softening occurs, however, the ability of the resin to support the fiber is severely reduced, along with the ability to transfer load through the matrix to the fibers. The result is a change in failure mode from filament dominated to matrix dominated.
- Fatigue damage often results in a significant reduction of strength and modulus of composites. The extent of damage varies widely, depending on the type of laminates,

nature of loading, etc. A longitudinal laminate (uniaxial tensile bearing element) shows hardly any strength change until immediately before failure. Whereas, a multi-directional laminate generally shows a gradual strength reduction until failure.

5.6.1 Mining-Concern Environmental Effects

Most of the applications of composite products in mining are limited to underground environment. Therefore, the damaging effects of elevated temperature and ultra-violet rays are of not concern. The potential source of damage could be moisture and acidic or corrosive chemicals.

Most often, adverse effects of moisture on composites are associated with elevated temperature, and available data is inseparable. This situation is not likely to occur in mining. The presence of moisture with and without acidic chemicals on the composite elements at mining temperature (i.e. 25 ± 5 °C) will not likely to cause any damage to them.

Similarly, most of the time, experiments concerning effect of moisture are carried out on layered composites. The thicknesses of layers are only few millimeters and a large number of ends of fibers is exposed to moisture. These test results cannot be generalized for tensile bearing tendons, exclusively produced for mining applications. The reason is that most of the fibers are arranged longitudinally and they are covered by special coating. Fibers are exposed only at one end of tendons, the other end of the bolt is usually covered with threaded section. Therefore, the amount of absorption of moisture is only through one end and effected area is expected to be the exposed end only.

Composite tendons can only be used as fully grouted tendons, however, with certain modifications, they might be used as mechanical anchors. In former case, tendons would remain in protected layers of grout, and in latter case, ends of tendons could easily be sealed with water (or moisture) proof sealant.

Dickson et al. (1984) exposured different composites in a sun-test cabinet for 2500 hours. They observed that the properties of cross-plied laminates of glass and Kevlar-49 epoxy-based composites remained substantially unaffected, even though long term exposure caused considerable darkening of the surface skin. These test results prove that occasional exposure to ultra violet rays do not damage composites.

5.7 Applications of Composite Materials in Mining

Composite materials have several potentially significant applications in mining. They have already been used in milling processes and in conveyer belts. The following are some of the potentially significant applications in mining:

1. As underground support structures, composite materials have the capacity to replace heavy steel structures like H or I section beams and columns. Moreover, composites can also be fabricated into rigid arches (curved H-sections), yieldable arches, posts and linear plates for shafts. Rock-bolts and cable-bolts (flexible tendons) of composite materials also have potential applications as interior rock support elements in mining. In addition, composites qualify to be used as ore pass liners to replace timber, steel, concrete or the various combinations of these materials currently used.
2. Material transport is one of the higher cost operations at the mine site. Haulage equipment such as trucks, scrapers, mine cars and shuttle cars are very heavy and expensive to operator. Composite materials could significantly reduce the weight and

increase the performance of haulage equipment. For example, the ratio of payload weight to truck tare weight could be increased from 1.5 to 2.0, with the result that a full 170-tonne truck would carry 30-tonne less dead weight and require 10 percent less horsepower. Similarly, filament wound drive shafts and suspension systems of reinforced plastic might save as much as 50 percent over conventional steel component weight (Owens et al., 1989).

3. The recent generation of robotics systems have heavy structural members fabricated from commercial metals. These members provide rigidity and structural integrity but generate large internal loads that inhibit their performance. For example, current robots are characterized by payload weight-to-arm weight ratios of only about 1:20. However, if a superior range of robotics products are to be developed for the mining industry, then high strength, light weight articulating structural members must be fabricated to increase payload, reduce energy consumption, improve the dynamic response of the system and increase the operating speed, at the same time ensuring the structural stiffness is not compromised (Owens et al., 1989).

4. The conveyor belt industry has already used high performance composite materials. Load bearing steel components have been replaced by thermoplastic and filament-wound components. In longwall tunnels, a chain conveyer is used in transportation. Productivity depends on undistributed long testing performance of the chain conveyer. The existing metal chain conveyer cannot be guaranteed against corrosion, blockage and economy (due to heavy weight). All the existing metal components like rails, wheels, transportation equipment etc. degrades due to corrosion in salt mines. Composite components work successfully at low cost in such salt mines in place of existing metals. A durable, light weight and economical chain conveyer can be fabricated from polyethylene thermoplastic composite material.

5. Underground storage rooms to store chemicals can be manufactured by fire and corrosion proof composites.

5.8 Civil Engineering Applications

The focus for more immediate application of composites in mining is in underground support. Therefore, all the examples reviewed here are from civil fields of relevance to support systems.

Fiber-reinforced prestressing tendons were used in the construction of Ulengergstrane Highway bridge in Dusseldorf, Germany, in 1986. The two-span slab bridge (max. dead load of 60 tonne per vehicle) has 21.3 m and 25.6 m (69.9 ft and 84.0 ft) spans. Bridge has been monitored since its opening for traffic and it has been working as a normal structure (Wolff and Miesseler, 1987).

Beams with a span of 9.0 m length and 60.0 cm height were prestressed by a 19-bar composite tendon with a permissible prestressing force of 600.0 kN (134.9 lbs). Test results showed that the beam was more ductile then the conventional prestressed version and it returned to its initial straight position after a full relaxation of the test load (Wolff and Miesseler, 1987).

In Japan, a pretensioned prestressed concrete slab highway bridge with a 5.76 m (18.9 ft) span and 7.0 m (22.9 ft) wide was constructed with carbon tendons (Minosaku, 1992). There are several other bridges constructed in Japan and Germany using composite tendons. The capacity and performance of these bridges are equivalent to concrete bridges.

Nani et al. (1992) investigated and determined the transfer length of composite tendons, made of braided epoxy-impregnated aramid fiber, embedded in concrete members. Ballinger (1990) documented applications of composites as structural components in a four 35 ft high by 35 ft square rooftop turrets that house radio antennae erected on glass fiber-reinforced I-shape. Bedard (1992) discussed highway bridges damaged by corrosion and composite reinforcing bars as a remedial measures.

Chapter 6

Composite Material Tendons

Extensive literature and market surveys were conducted to gather information on composite tendons that are or can be used in mining. After scrutinizing this information, sixteen different types of tendon were selected for further research and testing. The selection criterion was based on the mechanical and physical properties of the tendons, as well as their cost. Although tendons can be made of identical material, their properties frequently vary widely. Therefore, each patented composite tendon is discussed herein. The manufacturing materials and processes along with their mechanical, physical properties are discussed in this chapter. The advantages and disadvantages of composite tendons in mining and in general are highlighted. Some of the case studies and field or laboratory experiments involving composite tendons are also mentioned. The mining constraint experiments are carried out at McGill Rock Mechanics Laboratory and their detail is given in chapter 7 and onwards. The following are the types of composite tendons:

1. Arapree
2. Weldgrip fiberglass rockbolts
3. Weidmann fiberglass rockbolts
4. Polystal tendons
5. Polyglas
6. Celtite rockbolts
7. Kodiak fiberglass-reinforced plastic rebars
8. Isorod composite rockbolts
9. PSI fiberbar composite rockbolts

10. Thiessen fiberglass rockbolts
11. Kevlar
12. Extern fiberglass
13. Newport composites
14. Ryton-PPS
15. Fiberloc
16. Mincon

6.1 Arapree

Arapree is a braided epoxy impregnated aramid fiber. It is manufactured exclusively to be used in place of steel tendons in prestressing concrete industry. The combination of low density (1250 kg/m3) and high strength (2800 MPa) and high elastic modulus (125 GPa) makes Arapree an ideal choice for reinforcing potentially unstable rock mass. Arapree is available both in sanded and helical winded round shape and in rectangular strip shape with regular pattern of small knobs. Rectangular shape is more suitable for mining. Arapree can be used both in place of (flexible) steel cable bolt and (rigid) rockbolts. The cross-section dimension of Arapree is 20 x 5 mm and its breaking load is more than 13 tonnes.

ARAPREE is an ARAmid PREstressing Element made of Twaron continuous aramid fibers pre-impregnated in epoxy resins. The impregnation of Twaron fibers into epoxy resin resulted into Arapree tendons. The manufacturing process is braiding, in which, long fibers are pulled from different spindles and combined to a pre-selected and pre-designed pattern and then passed through orifice of selected dimensions. The orifice or opening determines the shape of the final product, for example, circular or rectangular. Then in order to obtain various properties, in addition to dimensional stability, the final product is impregnated into epoxy. In the final stage, various types of surface profile of products are obtained such as engraved or sanded surface, etc.

The combination of Twaron fibers and epoxy resin result in a composite product which has:

- High strength and high stiffness
- Low creep and relaxation
- High stability in many environments, such as sea water, oil, calcium hydroxide, chlorides
- Moderate thermal stability
- Low density
- Non-conductivity
- High tensile fatigue resistance

Physical Properties of Arapree

The following are the physical properties of Araprec HM:

Geometry

Fibers: -Aramid, type Twaron, endless

- Cross-section of filaments: round, diameter 12 x 10^{-3} mm.
- shape : rectangular and round, endless.
- type: round and strip like tendon, containing multiples of 100,000 Twaron filaments in epoxy resin, orthogonally anistropic.
- effective aramid cross-section: 11.1 mm^2 per 100,000 filaments

- composite cross section: n x [1.5 x 20]mm^2 = n x 30 mm^2
- surface pattern: sanded and regular pattern of small knobs.
- Volumetric composition 35 - 45 % Twaron HM; 60 - 55 % resin

Mechanical Properties of Arapree

Twaron fibers, based on stiffness, are manufactured in two grades; Twaron with modulus of elasticity in the range of 70 GPa and Twaron High Modulus (HM) with modulus of elasticity approximately 130 GPa. The data is referred to Arapree High Modulus for the reminder of the chapter. The mean values of mechanical properties of Arapree HM at 20° C are given in Table 6.1.

Table 6.1: Mechanical Properties of Arapree

Property	Units	Short Term	Long Term
Modulus of elasticity	GPa	125.0	125.0
Tensile Strength	MPa	2800	2400
Strain at Failure	%	2.24	1.92
Density	kg/m^3	1250	NA
Poisson's Ratio		0.38	NA

NA = Not available

Creep and Relaxation Behaviour

Creep and relaxation are interrelated material properties of a structural element and both are function of stress, strain and time. Creep can be described as change in strain as a function of time at constant stress. Relaxation is due to change in stress as a function of time at a constant strain. Figure 6.1 describes the concept and possible relation between creep and relaxation at a certain level (Gerritse, 1987). Since, steel is supposed to be replaced by aramid fibers, therefore steel is used as standard to compare various properties.

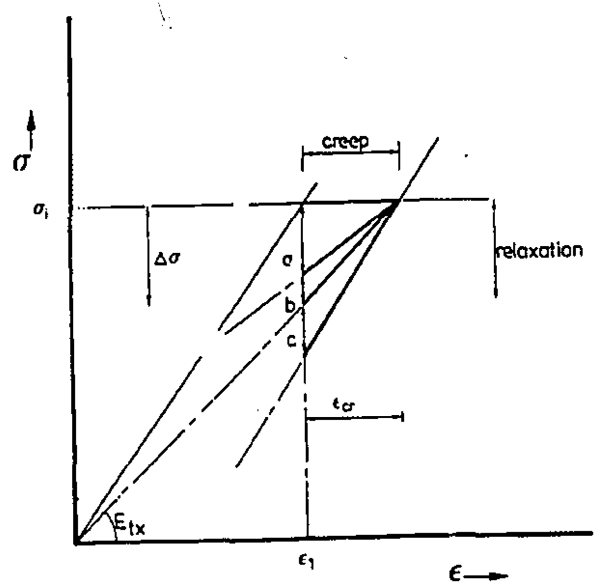
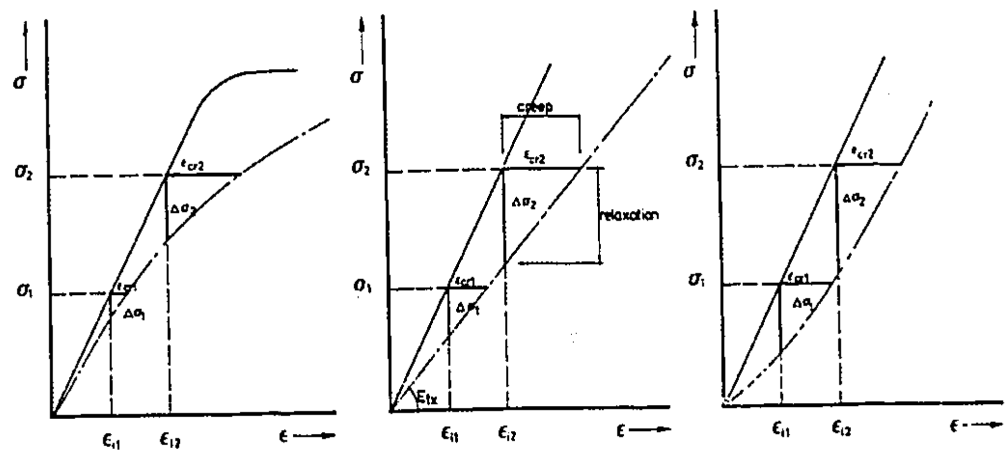


Figure 6.1: Possible relations between creep and relaxation (Gerritse, 1987)

Three possible relationships between creep and relaxation are given by Gerritse et al. (1987). If relaxation is simulated by a creep test whereby, after a certain time interval, the measured creep is compensated by a decrease in load, then the obtained relaxation will be according to a, b, c in Figure 6.2. Figure 6.2(a) reveals the behaviour of steel. This demonstrated that the initial stress level gives a more than proportional increase in creep strain ($\varepsilon_{cr2}/\varepsilon_{cr1} > \sigma_2/\sigma_1$). Relaxation data illustrates a comparable proportionally, from ε_{cr1} to ε_{cr2}. Conclusively, it can be said that contrary to prestressing steel - increase in initial stress produces constant or even decreasing relaxation percentages for aramids for the same time interval. $\Delta\sigma_2/\Delta\sigma_1 \leq \sigma_2/\sigma_1$; see Figure 6.2 (b & c).



(a) (b) (c)

Figure 6.2: Schematic creep/relaxation behaviour at different initial stress-levels (Gerritse, 1987)

Figure 6.3 shows experimental results conducted to evaluate the creep of Arapree. The relationship is approximately linear between applied stress and the creep strain. The values given are determined at an initial stress level of about 50 % of the characteristic strength. Moreover, the test results conducted on relaxation behaviour shows that polymeric materials are more or less independent of the applied stress level. Figures 6.4 and 6.5 show relaxation of Arapree in air and alkaline solution respectively.

Stress Rupture

The stress-rupture behaviour of Arapree both in air and Alkaline solution is graphically shown in Figure 6.6. The characteristic value in Figure is defined as the value below which 5 % of all possible test results may be expected to fail. The mean stress rupture level of Arapree in tension after 100 years is approximately 65 % of short term strength. A sustained stress in air at a level below 65 % of short term strength does not lead to a stress-rupture. Figure 6.6 shows this phenomenon (Gerritse and Werner, 1988).

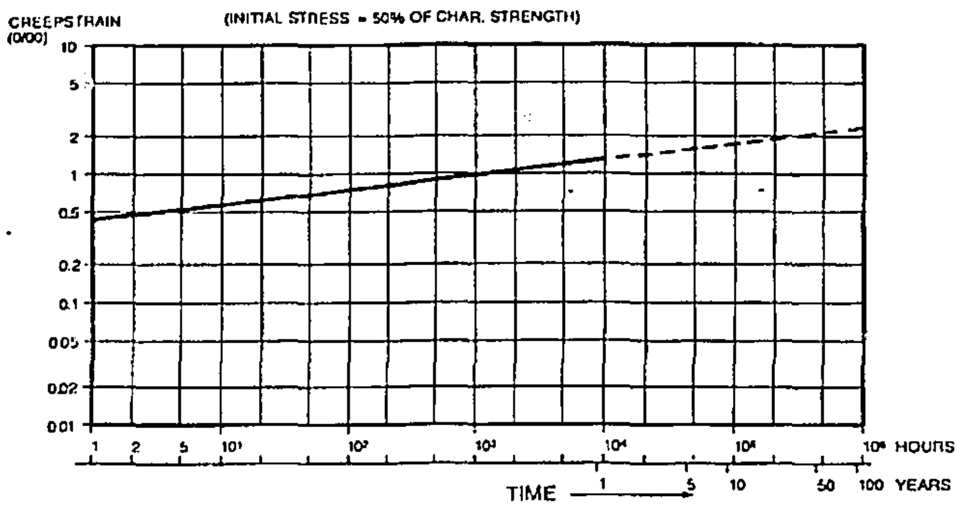


Figure 6.3: Creep behaviour of Arapree in air (Gerritse and Werner, 1988)

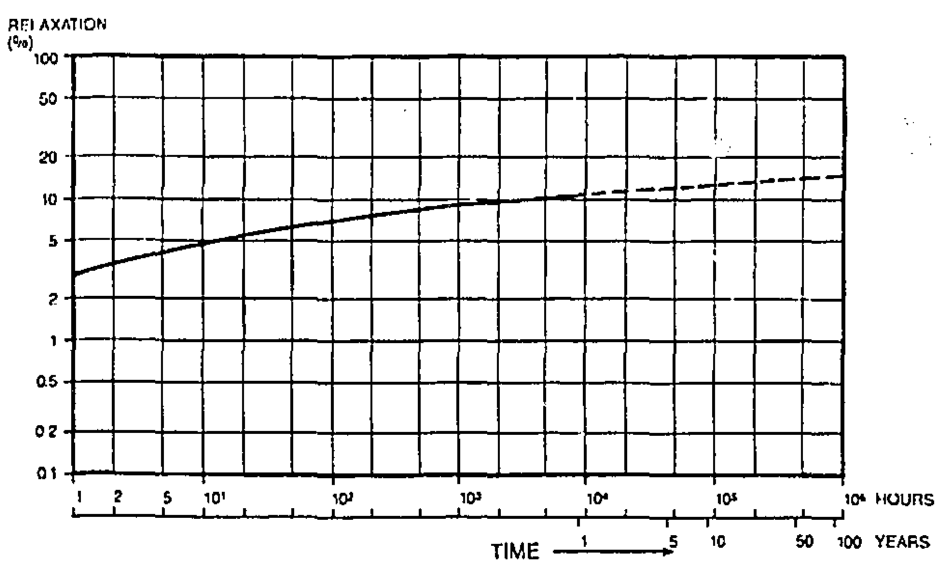


Figure 6.4: Relaxation of Arapree in air (Gerritse and Werner, 1988)

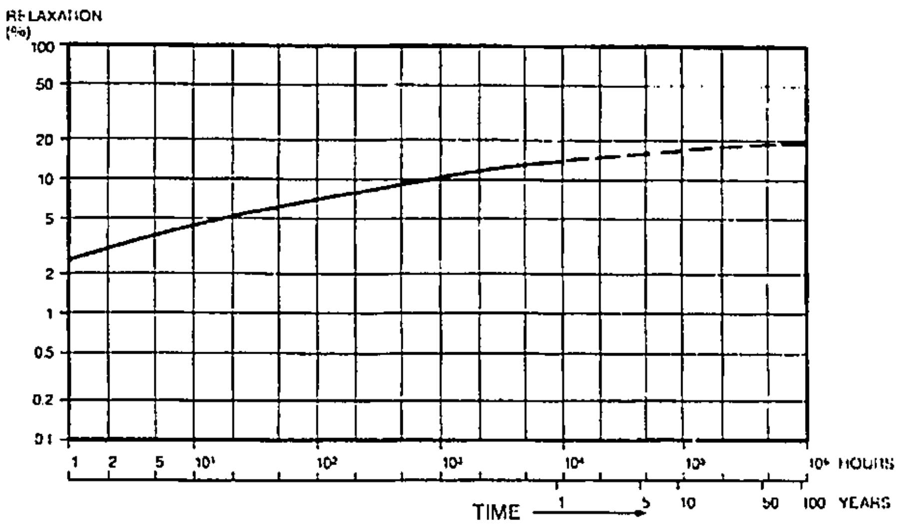


Figure 6.5: Relaxation of Arapree in alkaline solution (Gerritse and Werner, 1988)

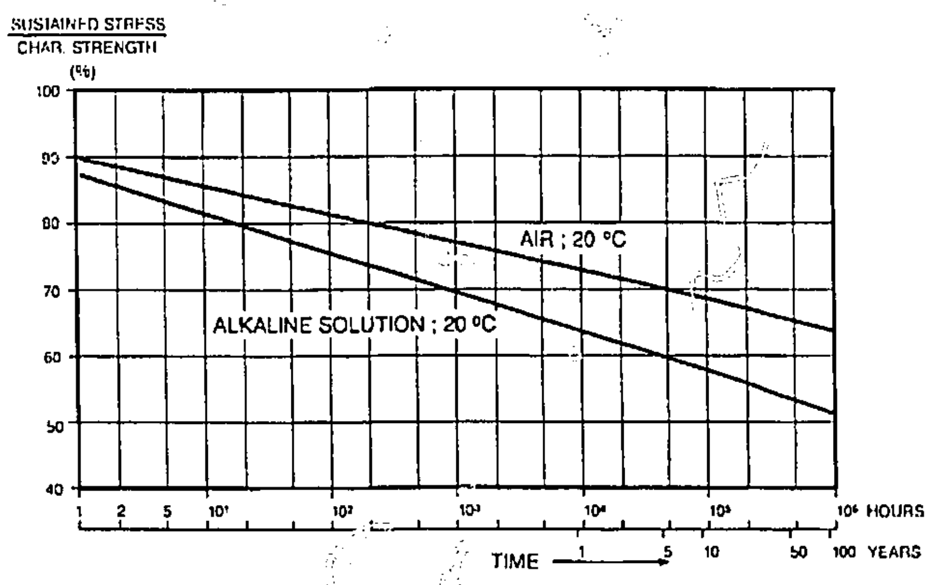


Figure 6.6: Stress-rupture behaviour of Arapree in air and alkaline solution (Gerritse and Werner, 1988)

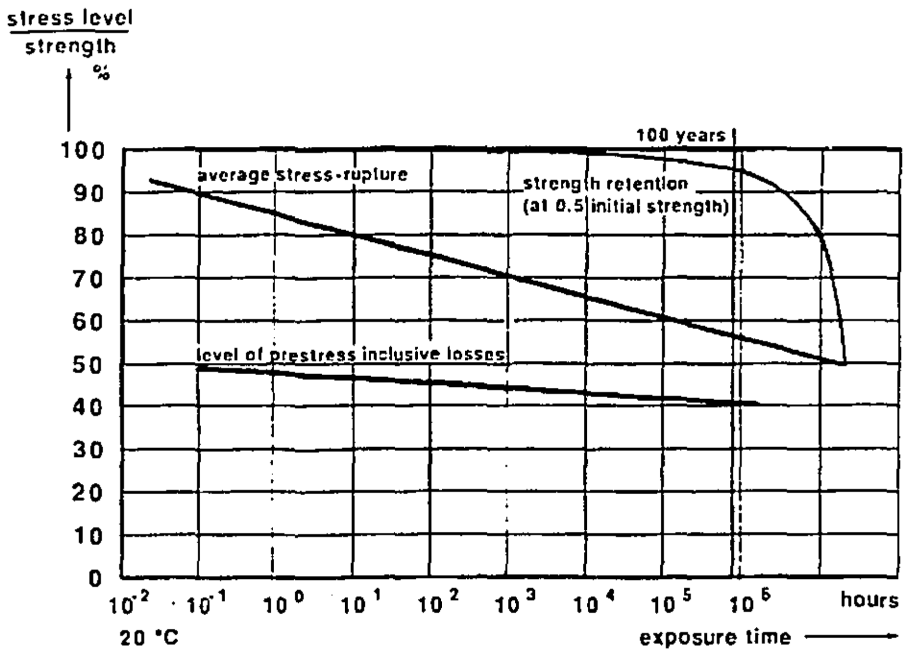


Figure 6.7: Safety prognosis Arapree in concrete (Gerritse and Werner, 1988)

Thermal and Electrical Properties

Aramid fibers at elevated temperature or under prolong heating behaves much better than other polymeric materials. The strength of Arapree increases at low temperature but without becoming brittle. The thermal expansion coefficient is -1.8×10^{-6} 1/^{o}K at 40 ^{o}C. The electric resistance of Arapree is 7×10^{15} ohm x cm in air and 7×10^{7} ohm x cm in water saturated environment.

Anchoring Methods

The ever-existing problem of all the non-metallic tendons is the incompatible holding or anchoring devices. The conventional anchoring devices (grips, chucks, etc.), made of steel, crushes the non-metallic testing samples. However, unlike glass-fiber reinforced tendons, Arapree does not get crushed due to line load (the anchoring load). It is also observed that a strip shape is practically easy to hold without undergoing slipping during testing as compared to round shape. This is one of the reason of producing Arapree in the strip shape (Gerritse and Werner, 1988). An anchoring device which is presently used in the experiments is shown in Figure 6.8. Its components are the outer steel cover and polymide inner wedges .

The tests conducted in Akzo laboratory to find the bonding strength between strip shape Arapree and concrete is shown in Figure 6.9. This figure also shows that an embedment length of 6 cm is enough to obtain breaking strength of strip Arapree. However, the testing done in mining constraints is presented in the forthcoming sections.

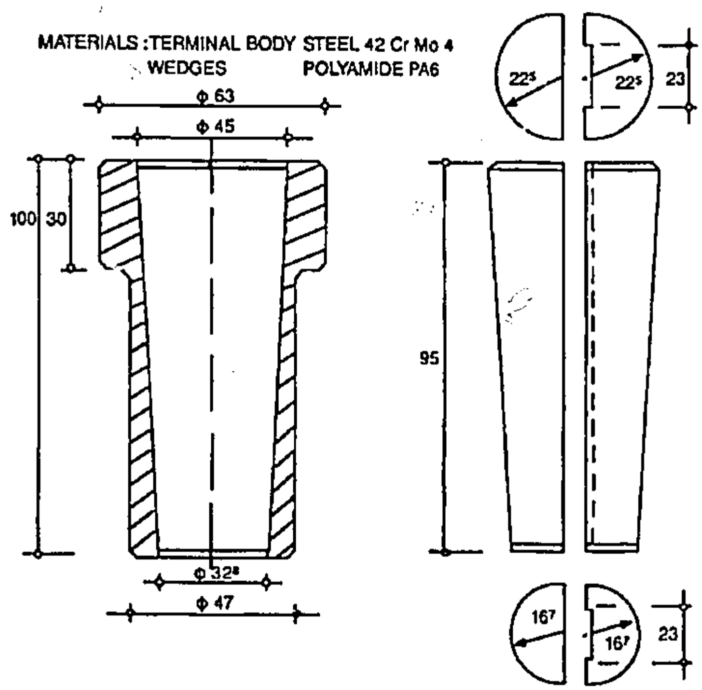


Figure 6.8: The anchoring device for strip shape Arapree (Gerritse and Werner, 1988)

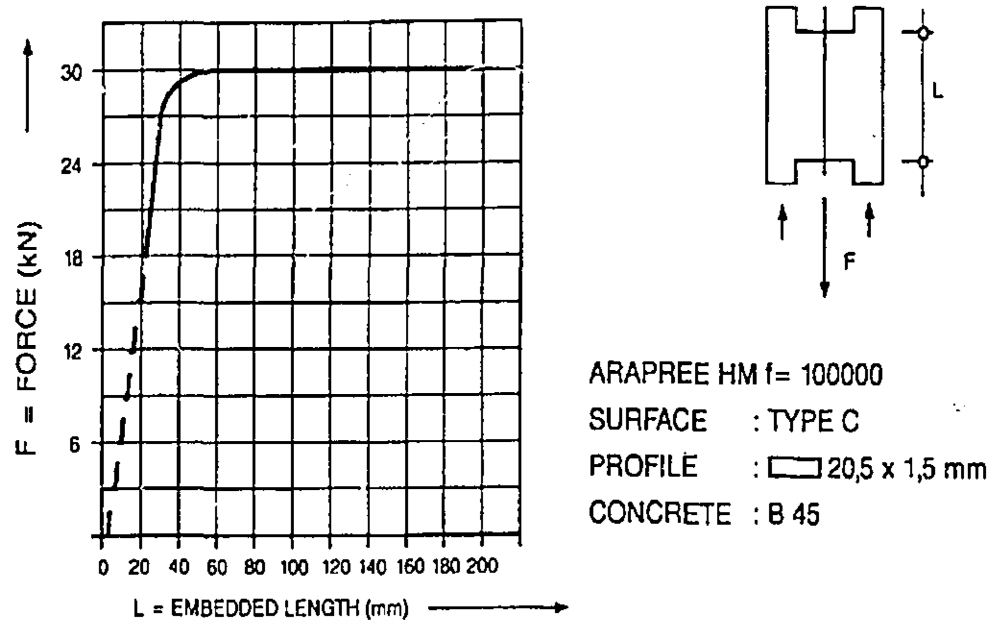


Figure 6.9: Critical embedment length of Arapree (Gerritse and Werner, 1988)

Another series of experiments on concrete elements was conducted in the Akzo laboratory to investigate the behaviour of Arapree in actual practice (Gerritse et al., 1986). Concrete planks were centrically and excentrically prestressed by Arapree tendons. The cross-sectional dimension of Arapree tendon was 0.25 x 20 mm. Figure 6.10 shows the loading pattern on planks and the deflection in planks due to this applied loading. In spite of very small lever arm of the vertically placed tendons, the tests proved that the tendons were ultimately stressed over the whole section at failure. The test results also showed an ideal pattern of finely divided cracks in bending failure. The slow propagation of such pattern of cracks demonstrated that adequate warning can be anticipated prior to the failure of structure supported by Arapree cables.

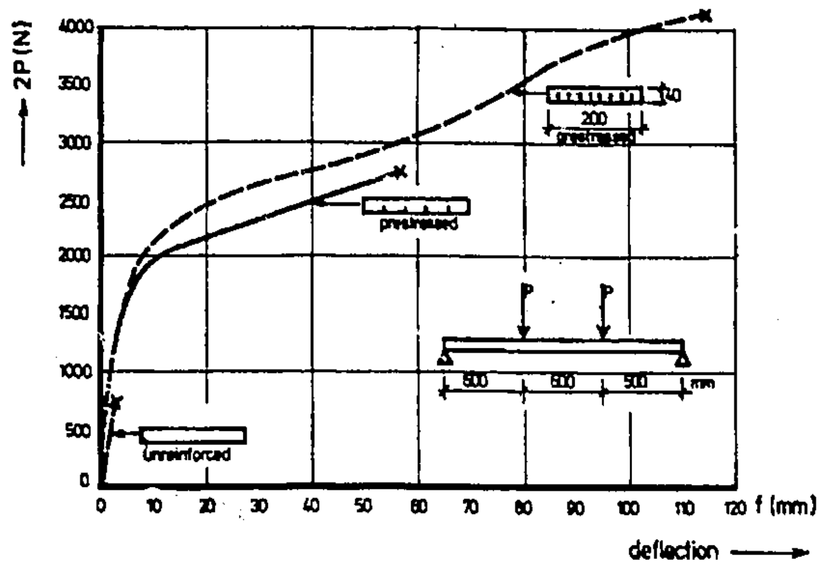


Figure 6.10: The applied loading on concrete plank and the resulting deformational behaviour, the cracking pattern is also visible (Gerritse et al., 1986)

Mining Applications

The general mining applications of Arapree tendons could be numerous. The basic use of Arapree is as tensile bearing element. Arapree tendons can be used both as rockbolts and cable bolts. Since, Arapree is flexible enough to be coiled in a diameter of approximately 1-meter, therefore, it can be used as flexible cable where enough head room is not available. Arapree cables can also be used as lacing; to hold loose rock pieces failed due to rock-skin-failure. The failure of pillar due to uniaxial compressive loading can be prevented or slow-down with the help of Arapree tendons. This can be achieved by winding the Arapree around the pillar. Such ideas as to continue Arapree cable from one drilled hole to another one (i.e. U-shaped configuration) is also possible. This technique will replace accessories such as nut, plate, lacing, etc. Arapree cables have the potential to pre-support cut-and-fill stope.

Table 6.2: Properties of Arapree

Arapree Type	Shape	Size (mm)	x-sectional area (mm^2)	Braking load (kN)
f 100,000	Round	3.9	22.2	33.3
f 200,000	Round	7.9	22.2	66.6
f 100,000	Rectangular	1.5 x 20	11.1	33.3
f 200,000	Rectangular	2.6 x 20	22.2	66.6
f 400,000	Rectangular	5.0 x 20	44.4	133.2

6.2 Weldgrip Fiberglass Rockbolt

Introduction

These are manufactured specifically as rockbolts and their potential imminent use is in UK coal mines. The load required to break a 25 mm diameter solid Weldgrip rockbolt is more than 30 tonnes. The surface profile is very rough with engraved knobs. These properties make this product also potentially ideal for bolts in continuous mining in hardrock mines. In addition to high strength, Weldgrip bolts are of high E-modulus, anti-static, non-corrosive and dimensionally stable; and come with all accessories such as end-plate, nut, bolt, etc. Weldgrip bolts are manufactured by pultrusion processes with glass fibers and polyester resin being the main constituents.

Weldgrip Fiberglass Rockbolts (WFR) are manufactures exclusively as rockbolts. Their anticipated applications were in coal mines to replace wooden dowels and steel bolts. However on the basis of their performance in laboratory experiments, a wide range of applications in hard rock mining can be predicted (Daws, 1992).

WFR are manufactured by the Pultrusion process. The chief constituents are continuous fibers of glass and polyester resin. Tensile load is mainly taken by fiberglass, whereas resin serves two purposes (1) keeps the fibers together in the specified shape and (2) takes the compressive load.

Advantages

1. Tailored Surface Profile

WFR can be classified as advanced fiberglass rockbolts because of their very rough surface profile. This was seldom possible in the pultrusion process a few years ago. This is a breakthrough in the technology and can be benefited if proper attention and research is devoted towards this field. The present rough surface profile is created by the manufactures not by the mining engineers. More engineering should be devoted in designing and laboratory experiments to produce an optimum designing of rough surface of bolts. This leads to the concept of compatible ductility and surface profile of bolts that should be suitable with different rock mass conditions, insitu stresses, discontinuities, local and global deformation of opening, and creep, shrinkage and dilation of rock.

2. Threaded Portion for Bolt and Plate

A special technique is used to encapsulate threaded portion to accommodate bolt and plate (or straps). This is provided after or during manufacturing process using a different material. The tensile thread testing shows that this thread portion can take uptil 20 tonne of load.

3. Like other fiberglass rockbolts, WFR are high strength, light weight, dimensionally stable, non-corrosive, non-conductive and cuttable.

Disadvantages

Possible applications are limited to rockbolts only. Due to their stiffness, WFR cannot be used as cables. The ductility of bolts plays an important role in the performance of fully grouted rockbolts under dynamic loading. This factor is yet to be investigated

Anchoring Capacity

Laboratory testing was done to determine the bonding characteristics of the rod profile and the strength of the moulded threads. All these tests were undertaken within the Laboratories of the Department of Mining Engineering at the University of Nottingham (personal communication, 1992).

Bonding Characteristics

Four rods were used to determine the bonding characteristics. These rods were grouted into a sandstone block with a bonded length of 300 mm. Holes of 27 mm diameter were pre-drilled to the required length and each rod was grouted into the position using Resifix 35 pourable Resin Anchor Grout. These rods were left for 48 hours before starting the tests.

At the free end of each bolt two steel clamps were positioned and a direct load was applied using an Enerpac hydraulic hollow cylinder ram of 60 tonne capacity. Pressure supplied to the ram was by the Enerpac hydraulic pump.

Each steel clamp, measuring 4 inches in length, was fastened to the rod using 4 X 10 mm bolts and tightened using a pneumatic impact wrench, thus giving a total of 8 inches of gripped length. Load was applied and at a measured load of 20 tonnes the steel clamps slipped. On investigating the mouth of the hole it was apparent that no movement had taken place in this region and each rod exhibited perfect bonding characteristics.

Tensile Thread Testing

This test was undertaken using an Avery/Denison Universal Testing Machine of 500 tonne capacity. Twelve separate tests were undertaken where a direct load was applied to a pre-designed testing frame housing each bolt in turn. The load was applied to the threaded position through a steel washer plate held in position by the 30 mm retaining nut. At the lower end of the rod a steel clamp measuring 4 inches in length was fastened to the rod, tightened, using a pneumatic impact wrench. The applied load was evenly distributed over the top of the testing frame which measured 150 mm X 150 mm. A loading rate of 5 mm/min was used throughout the twelve tests. The results of these tests have indicated that the average failure load was 69 kN with a residual strength of approximately 30 kN over the length of the test. A graph confirming the average results of these tests was shown in Figure 6.11

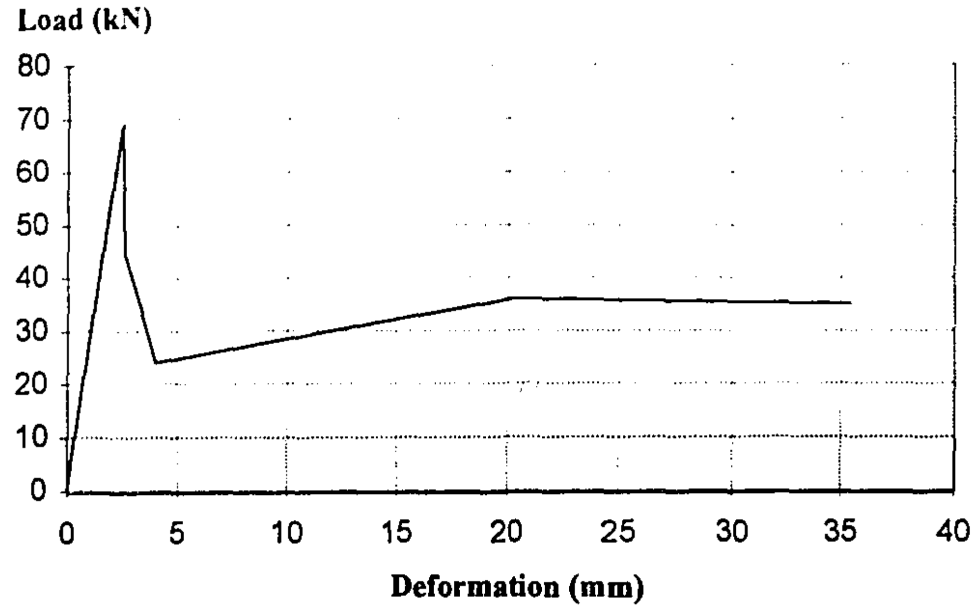


Figure 6.11: Load-deformation curve of pull out test of Weldgrip fiberglass rockbolts

6.3 Weidmann Fiberglass Rockbolt

Background

Weidmann products are fiberglass reinforced thermoplastic and thermosetting plastics. Weidmann rockbolts are developed exclusively to be used as cuttable rockbolts in mining support systems. Rod breaking load ranges from 160 kN to 350 kN. Weidmann rockbolts have been developed with all parts, such as thread, nut and plate, manufactured from fiberglass reinforced plastic. The thread is fitted to the rod by means of a patented manufacturing process to ensure maximum transmission of the load bearing capacity. This development eliminates the corrosion drawbacks of the metallic bolt; it is also cuttable and the low material weight makes handling and transportation easy. Design parameters and an installation method are recommended by the Weidmann company, however, there seems to be little remedy for the problems relating to the smooth surface profile. Weidmann rockbolts have only been used in soft rock mines, e.g. coal mines. Cost of 22 mm diameter rod is C $ 20.0 per meter (Weidmann's technical brochure).

Advantages

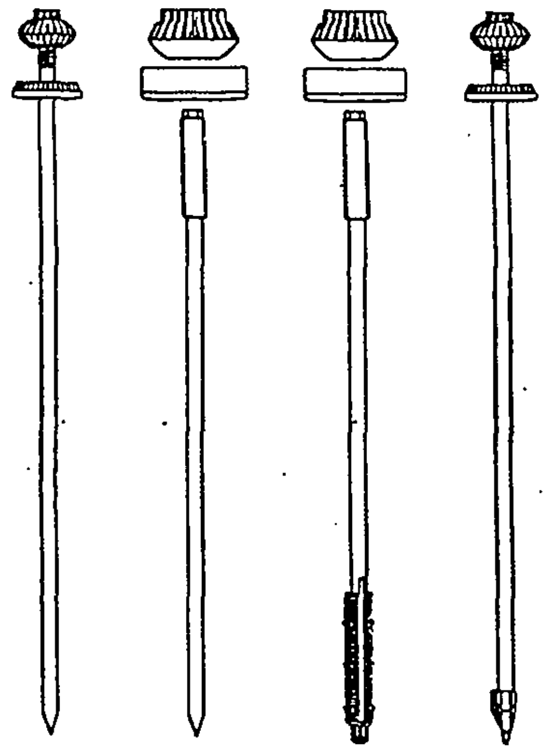
Weidmann Fiberglass Rockbolts (WFB) have already been used as rockbolts in some European coal mines. Anchoring and installation problems have been solved to some extent and WFB can be applied in weak rocks with confidence. Design guidelines are available to assist proper installation. In addition, fiberglass accessories such as nuts and plates, washers, holding plates for wiremesh, etc., are also available.

Disadvantages

Weidmann rockbolts are expensive compared to steel rockbolts. Proper installation cannot be assured despite careful attempts. The plastic rock plate is weak and cannot be relied upon under heavy load.

Mechanical Properties

Table 6.3 lists the mechanical properties of Weidmann fiberglass rockbolts.



	Solid Fiberglass Rockbolt		Hollow Core Fiberglass Rockbolt	
Type	KO	K2	JO	J2
Rod breaking load	350 kN	350 kN	160 kN	160 kN
Rod outer diameter	22 mm	22 mm	20 mm	20 mm
Max. load carrying capacity of bolted head	40 kN	100 kN	100 kN	40 kN
Rod bending radius	2 m	2 m	1.5 m	1.5 m

Table 6.3: Mechanical Properties of different types of Weidmann Fiberglass Rockbolts and the corresponding figures (Weidmann's technical brochure)

Case Study

Coal Mines in the Ruhr Area, Fed. Rep. Germany

During the development of drifts at the Niederberg Coal Mine, a combination of rockbolts and wiremesh were used as the sole support lining. The coal seams are situated at a depth of 500 - 800 m, the cross sectional area of headings was 15 - 19 m^2. Rockbolts having a length of 2.1 m were set in polyester resin (cartridge) over their full length. During the later coal-extraction by shearer or roadheader, the Weidmann rock bolts were easily cut without causing any damage to cuttertools (Weidmann's technical brochure).

6.4 Polystal

Introduction

The basic constituents of Polystal (composite) are approximately 80 % by weight E-glass reinforcing fibers and high grade polyester resin. The orientation of reinforcing fibers is strictly unidirectional (longitudinal) and combined with thermosetting plastics to form higher quality semi-finished profiles. Polystal tendons have high tensile strength, which qualifies it as a durable structural support for pre- and post- tensioning of concrete and reinforcing rock in underground mining. Polystal tendons are chemical, thermal and corrosion-resistant. Polystal with 68 % glassfiber costs C $ 2.25 per meter for a 7 mm diameter tendon. Data and information have been derived from Technical Brochure of Polystal.

Advantages

High quality, controlled manufacturing processes could result in tendons with the following properties:

- Low density.
- Very high strength and stiffness.
- Excellent static and dynamic load bearing capacity.
- Outstanding electrical properties and,
- Excellent corrosive resistance.

A special wrap-around technique significantly improves the transverse strength and surface profile (rigid) of Polystal tendons. The handling properties and climatic stability are greatly enhanced by the high resin content of its surface.

Disadvantages

Like other components, the surface profile of polystal does not exhibit enough resistance to have an effective bond strength between grout and tendon. To overcome this problem, more than one tendon technique such as the birdcage, button cable or wrap-around technique seems necessary. This would complicate the installation problem and increase the cost of Polystal cable-bolts.

The other disadvantage associated with Polystal is the noxious gas which is evolved in the process of failure of resin and fibre contents. In civil engineering and the open environment this is not a serious problem, as harmful gases are rapidly dispersed. However, in the underground mine environment this could create hazards and legislation might not permit the use of such material.

Applications in Mining

Further improvements to make Polystal tendons environmentally safe would enhance its chances of application in mining as a tensile load bearing structural element. A birdcage technique (and others) can be adopted to increase the bond strength and transverse shear of Polystal cable-bolts. Furthermore, the installation problems arising due to such birdcage techniques could be eliminated by using a larger diameter Polystal tendon with the wrap-around technique which ensures high tensile strength and bond strength. It is not necessary that the tensile strength of the composite cable should be equal or more than that of steel cables, but the tensile strength must be greater than the maximum bond strength between cable and grout and grout and rock. In other words the bond should fail rather than the cable.

Mechanical Properties

Tables 6.4 and 6.5 show the properties of different types of standard tendons. Table 6.4 shows polypropylene covered Polystal U and Table 6.5 presents Polystal U.

Table 6.4: Properties of Polystal B tendons (polypropylene-covered Polystal U)

Type	Diameter d (mm)	Diameter D (mm)	Weight (g/m)	Tensile Breaking Load (kN)
V 48 B/ 2.5	1.95	2.50	7.8	4.0
V 48 B/ 3.0	1.95	3.00	9.3	4.0
V 96 B/ 5.0	2.70	5.05	24.0	8.0
V 432 B/ 9.0	5.75	8.96	86.0	36.0
V 744 B/ 11.0	7.45	11.00	135.0	60.0

Table 6.5: Properties of Polystal U tendons

Type	Diameter (mm)	Weight (g/m)	Tensile Breaking Load (kN)
V 6U	0.70	0.8	0.5
V 12U	0.95	1.5	1.0
V 24U	1.35	3.0	2.0
V 30U	1.50	3.8	2.5
V 48U	1.95	6.2	4.0
V 96U	2.70	12.0	8.0
V 432U	5.75	54.0	36.0
V 744U	7.45	92.0	60.0
V 1824U	11.8	222.5	152.0
V 3336U	16.0	422.0	278.0
V 6000U	21.7	750.0	490.0

6.5 Polyglas

Introduction

Polyglas is made from reinforcing a polyester resin with multi-strands of glass filament and alternating layers of glass material. The glass is drawn through the liquid resin which coats and saturates the fibers. Structural shapes such as angles, channels, I-beams, and round and square solid and hollow tendons are easily pultruded with Polyglas. The Polyglas ultimate tensile strength is 210 MPa with a 50 GPa of tensile modulus. Polyglas tendons have a smooth surface profile which leads to a weak bonding between the grout and rockbolt interface. A modification would therefore be required, either in the surface profile of tendons or in the installation techniques. Polyglas structural shapes offer resistance to many environmental hazards such as rot, rust, high temperature and atmospheric pollutants. The composites can be formed into a number of shapes through the pultrusion process. Polyglas rods are manufactured in three standards grades. The characteristics of each grade are determined by the shape of resin used and the proportion of glass filament in the matrix. The cost of 25 mm diameter solid rod is C $ 4.00 per meter.

Polymer General purpose grade used up to 155 °C.

Supersil High temperature grade for use up to 200 °C and beyond.

EHS - Extra High Strength This grade provides very high flexural strength (690 MPa (100,000 psi)), tensile strength and modulus.

Advantages

The following are the advantages of circular solid and hollow tubes of varying dimensions.

1. Strength

Tensile, compressive and flexural strength and modulus of Polyglas are comparable with other composites and in some cases exceed other forms of fiberglass reinforced plastic.

2. Weight

Polyglas is 25 % the density of steel and 40 % lighter than the aluminum.

3. Dimensional Stability

1. Solid and hollow rods of different length (up to 7 metres) and diameters are available commercially. Rods of other specification (custom-made) can also be pultruded, but it would increase the cost.
2. Coefficient of thermal expansion is lower than the other plastics.
3. Low moisture absorption, it does not swell when exposed to moisture.

4. Environment Proof

1. Polyglas has outstanding resistance against rot, rust, acids, bases and salts.
2. It maintains strength and flexibility at high and low temperatures.

Disadvantages

Polyglas rods have a smooth surface profile and this surface cannot be altered (since manufactured through pultrusion). The bond strength between grout and Polyglas rock-bolts cannot be predicted at this time in the research. A thorough study would be required. Polyglas has never been used in concrete or underground mining applications, so it is too early to predict its behaviour in such an environment. It is non-magnetic and therefore would be difficult to separate from ore in milling processes.

Mechanical Properties

Table 6.6 shows that the mechanical properties of Polyglas rods (not other structural shapes).

Table 6.6: Mechanical Properties Polyglas M Tendons (Polyglas Technical Brochure)

Property	Test Method	Direction	Typical Value
Ultimate Tensile Strength, MPa	ASTM D-638	Longitudinal	210.0
	ASTM D-638	Transverse	50.0
Tensile Modulus, MPa	ASTM D-638	Longitudinal	17237.5
	ASTM D-638	Transverse	5516.0
Ultimate Compressive Strength, MPa	ASTM D-695	Longitudinal	217.0
	ASTM D-695	Transverse	104.0
Compressive Modulus, MPa	ASTM D-695	Longitudinal	17237.5
	ASTM D-695	Transverse	6895.0
Ultimate Flexural Strength, MPa	ASTM D-790	Longitudinal	210.0
	ASTM D-790	Transverse	70.0
Flexural Modulus, MPa	ASTM D-790	Longitudinal	11032
	ASTM D-790	Transverse	5516
Density, kg/m^3	ASTM D-792	Longitudinal	1716

Applications in Mining

Polyglas rods can be used in place of steel rock-bolts. Because of its high strength and low modulus, Polyglas rock-bolts could sustain the small deformations which occur immediately upon excavation without breaking. Moreover, due to this property there are less chances of local failure of (caused by joint, fissures, etc.) in the immediate vicinity of (1 - 3 metres) of the tunnel opening. Polyglas is a light material therefore transportation, handling and installation is easier than steel rock-bolts. Different types of resins are used successfully as a binding matrix in mining support system. Ground control resins seem to have a compatibility with polyglas tendons, hence an effective bonding could be achieved between the rock-surface and Polyglas rock-bolts. Cement grout with Polyglas has not yet been tried in either the laboratory or the field, so it is premature to comment on its potential application.

6.6 Celtite Rockbolt

Introduction

Celtite rockbolts are a compound of fiberglass reinforced thermoplastic and polyester resins. Celtite rockbolts have high strength and stiffness, creep and fatigue resistance, with a low coefficient of expansion and dimensional stability. In field tests, a maximum of 100 kN pull-out load was achieved with both resin and cement grouts, whereas the steel rockbolt pull out load was 125 kN (Lapointe, 1990). The main disadvantage of Celtite rockbolts appears to derive from the associated loose fibers which cause skin irritation and can be inhaled during handling and installation. For a complete set including rockbolt (26 mm diameter), nut and plate is C $ 14.0 per meter

Advantages

Celtite fiberglass reinforced composites have outstanding properties like strength and stiffness, creep and fatigue resistance, low coefficient of expansion and dimensional stability. On the basis of these properties, Celtite rockbolts can be used as rockbolts to sustain the load applied during the deformation of freshly excavated rock. Celtite comes in different pultruded and molded shapes. Celtite rockbolts have been tested in underground mining and the results indicate that there is a definite potential for Celtite rockbolts to support structurally related failures. These rockbolts are chemically resistant, light weight and cuttable.

Disadvantages

Celtite rockbolts have very loose fiberglass fibers in the perimeter of the bolts. These loose fibers create two safety problems: 1) loose fibers can be inhaled during handling and installations 2) loose fibers cause skin irritation, therefore special gloves are required during the installations of Celtite rockbolts.

Field Test Procedure

Celtite fiberglass reinforced, cuttable rockbolts have been tested in underground mines in place of steel rockbolts. Holes were drilled in the intact moderate hardrock approximately 76 cm long and 3.2 cm in diameter. A piece of wood was inserted in all the drilled holes to obtain a uniform 30 cm embedded length (anchoring length). In addition to the Celtite rockbolts, steel rockbolts (threaded) were also tested in the same environment to compare the results. The diameter of steel rockbolts was 20 mm, whereas Celtite rockbolts had 20 mm and 26 mm diameters. Ground Control Polyester resins and cement were used as grouts. The curing time was 24 and 36 hours for resin and cement respectively. More than 30 holes were drilled, out of which 20 were selected for the experiment. All holes were drilled horizontally approximately 1 m above ground level (Lapointe, 1990).

Test Results

In the pullout tests the rate of loading was kept constant while displacement was varying. The diameter of drilled hole (32 mm) was not found appropriate for Celtite rockbolts of

20 mm diameter, because of the fact that there was more grout in between the rockbolt and the rock. This single factor seems to have lowered the ultimate pullout load resistance of these rockbolts. Encouraging results were found in the case of Celtite rockbolts with cement grout. This combination gave similar ultimate load carrying capacities as were found with steel rockbolts. In some instances cement grout is not considered suitable for other fiberglass rockbolts (like Weidmann fiberglass rockbolts). In all cases of Celtite rockbolts the displacement was comparatively more than the steel rockbolts as shown from Figures 6.12 and 6.13. Table 6.7 lists the mechanical properties of Celtite rockbolts. Table 6.8 shows the parameters used in the experiment (Lapointe, 1990).

Table 6.7: Mechanical Properties of Celtite Rockbolts (Lapointe, 1990)

Properties	Longitudinal	Transverse
Tensile Strength, MPa	140 - 700	30 - 70
Modules of Elasticity, GPa	14 - 40	4 - 8
Ultimate Elongation %	2	2
Flexural Strength, MPa	200 - 900	40 - 100
Compressive Strength, MPa	150 - 400	70 - 100
Impact Kg cm/cm^2	80 - 125	60 - 100

Table 6.8: Parameters used in the experiment (Lapointe, 1990)

Type of Rockbolt	Type of Grout	Rockbolt Dia (mm)	Anchoring Length (mm)	Drilled hole Dia (mm)
Steel	Polyester Resin	19.5	320	32
Celtite	Polyester Resin	20	348	32
Celtite	Polyester Resin	26	315	32
Celtite	Cement	26	295	32

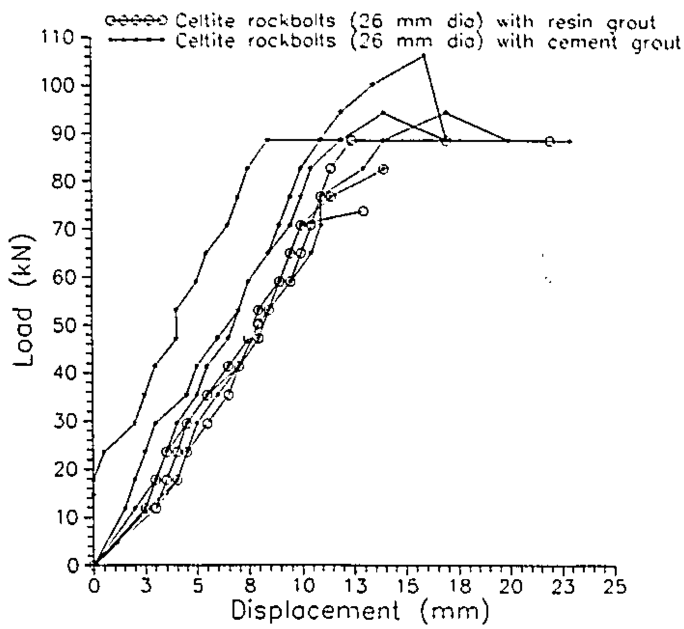


Figure 6.12: Pull out behaviour of Celtite rockbolts grouted with cement and resin grouts (Lapointe, 1990)

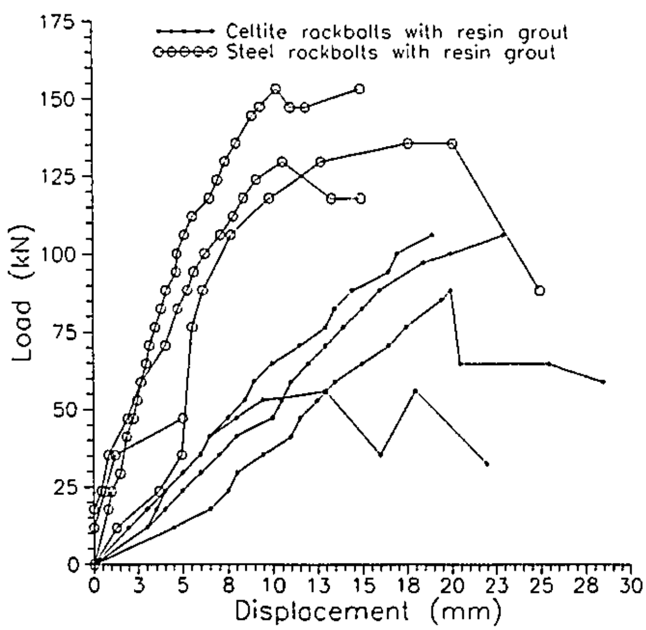


Figure 6.13: Pull out behaviour of Celtite and steel rockbolts (Lapointe, 1990)

6.7 Kodiak Fiberglass-Reinforced Plastic Rebar

Introduction

These bars are another product of fiberglass and resin. They were developed to be used in the concrete industry to replace existing deformed steel bars. In the manufacturing process, a band of glass fibers is wound around Kodiak FRP in a spiral shape to create a "deformed" surface. It is expected that this deformed shape would provide the same rebar-to-grout bonding as obtained in the existing steel deformed rebar-to-concrete bond strength. The cost of 25.4 mm diameter solid rod is C$ 6 per meter.

Typical Application

- Hospital magnetic resonance imaging facilities
- Radar installations
- Electricity generating and transmission facilities
- Marine structures exposed to salt water
- Chemical plants and storage sites
- Architectural concrete
- Tilt walls where rust from steel rebar might discolor the surface

Advantages

Fiberglass-reinforced-plastic rebar, like other FRP products, offers important advantages over its steel counterpart. Two of these advantages are:

1. Fiberglass-reinforced plastic is non magnetic and electrically non conducting.
2. It will not rust and is immune to electrolytic corrosion and to a wide range of acids, salts, and other chemicals that attack steel.

Because of these two advantages, Kodiak FRP rebar has gained acceptance for use in reinforced concrete applications where conventional steel-reinforced concrete would be unacceptable. The first advantage makes it possible to use reinforced concrete in areas where electrical or magnetic interference is undesirable; for example near magnetic resonance imaging systems and radar stations. And, because it is non conducting, Kodiak FRP rebar is finding increased application in electrical generating and transmission facilities. Its corrosion resistance makes Kodiak FRP rebar preferable to steel for marine structures exposed to salt water and for chemical plants and similar installations.

Yet another advantage FRP has over steel is a high strength-to-weight ratio. FRP rebar has twice the tensile strength of steel at one-fourth of the weight. Although this property is less important for reinforcing concrete than for some other applications (fiberglass grating, for example), Kodiak rebar is frequently used in architectural (non structural) concrete castings because it is light and easy to handle and will not rust.

Low temperatures do not affect FRP rebar, which has been tested for brittleness at -51°C (-60°F); there was no change. FRP rebar exhibits higher strength characteristics at low temperatures. High temperatures become a factor if the members are exposed to temperatures above 110 °C (230 °F) for extended periods. For example, at 149 °C (300°F), tensile strength is reduced by 10 percent, flexural modulus by 33 percent Kodiak Technical Brochure.

Disadvantage

As mentioned before, this type of rock bolts has never been tried in underground mining environment or in the laboratory to find out all the design parameters. Kodiak FRP is rigid and can not deform. This suggests that a brittle failure of this type of bolts can be resulted under dynamic loading situations (rockburst, underground blasting, etc.). A thorough research is required to establish grout-bolt failure and compatible installation techniques.

Applications in Mining

The use of Kodiak FRP in underground mining would appear to be rock-bolts only.

Mechanical Properties

The typical stress-strain diagram for FRP rebar is a straight line to almost the point of failure. The yield point is 3 to 4 percent before the break. Table 6.9 shows the available stock size and Table 6.10 shows the mechanical properties of Kodiak FRP.

Table 6.9: Available stock sizes of Kodiak FRP (Kodiak Technical Brochure)

Nominal Diameter, mm	Nominal cross-section, mm^2	Weight per linear m, Kg
9.2	66.03	0.013
12.25	120.05	0.023
15.31	186.08	0.036
18.37	264.1	0.051
21.44	360.15	0.069
24.5	474.2	0.090

Table 6.10: Mechanical properties of Kodiak FRP* (Kodiak Technical Brochure)

Property	Value
Ultimate tensile strength, MPa	689.5
Tensile modulus of elasticity, GPa	41 to 50
Compressive modulus of elasticity GPa	50
Compressive strength, MPa	413
Bond strength, MPa	8.27
Shear strength , MPa	58.6
Coefficient of thermal expansion	5.2×10^{-6} mm/mm/°C
Yield strength, MPa	689.5
Water absorption	0.25% maximum
Specific gravity	2.0
Density, Kg/m^3	0.074

*15.3 mm rebar and 30 MPa concrete were used for test.

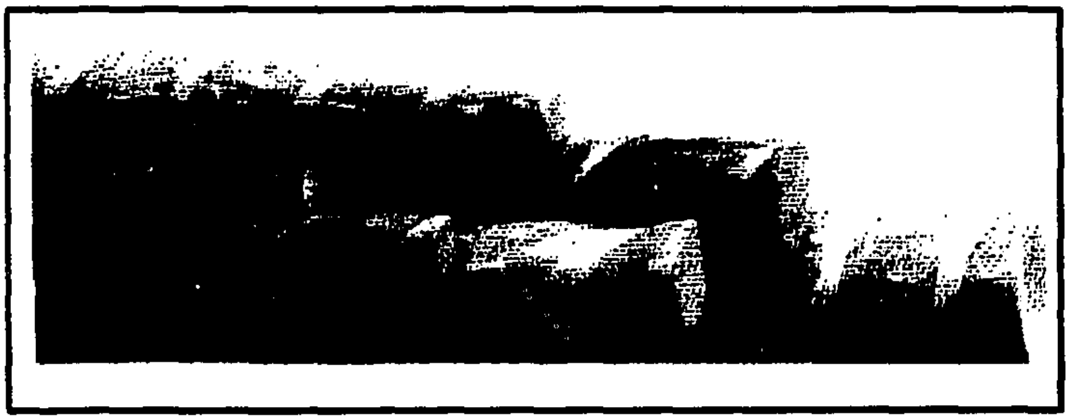


Figure 6.15: Stock of different sizes of Kodiak FRP, spiral surface profile is shown (Kodiak Technical Brochure)

6.8 Isorod & PSI Fiberbar Composite Rockbolt

Introduction

Isorod & PSI Fiberbar are fiber-reinforced plastic (FRP) i.e. rods of glass fibers in a polyester resin produced by pultrusion, a manufacturing process in which continuous fibers are pulled through a resin bath and then a heated die, thereby producing structural shapes of constant cross section. Isorod FRP rods are manufactured in Canada while PSI Fiberbar are product of USA. These products have been specifically developed as reinforcing bars for concrete construction.

Applications in Mining

Their applications are limited to rockbolts to reinforce rock structures. Figure 6.16 and 6.17 shows that efforts have been made to change and improve surface profile of composite bolts to obtain a design bonding strength between grout and bolts. Manufacturing of this type of surface roughness is an achievement toward the concept of cuttable support system.

Mechanical properties

Table 6.11 compares basic properties of composite reinforcing bars with those of ordinary deformed steel. Note that pultruded rod properties are strongly anistropic, with highest values obtained along fibers.

Chemical properties

The greatest advantage of Isorod & PSI Fiberbar composite rockbolts over steel bars is high resistance to chemical attack from acids, alkalis, solvents, oils, etc. under a wide range of temperatures. In particular, composite reinforcing bars with no additional protection appear totally immune to corrosion induced by natural exposure or common agents like sea water and deicing salts.

Electromagnetic properties

Unlike steel, FRP composites behave electrically as insulators. This property is invaluable in applications where reinforced structures are located close to high tension wires or where electromagnetic neutrality is a requirement.

Table 6.11: Mechanical properties of Isorod & PSI Fiberbar (Bedard, 1992)

Products	Ultimate Tensile Strength, MPa	Ultimate Compressive Strength, MPa	Modulus of Elasticity, GPa	Specific gravity
PSI Fiberbar	1034		41.4	1.86
Isorod	700	600	44.8 (tensile) 34 (compressive)	1.5 - 2.0
Deformed Steel Bars	480 - 690		200	7

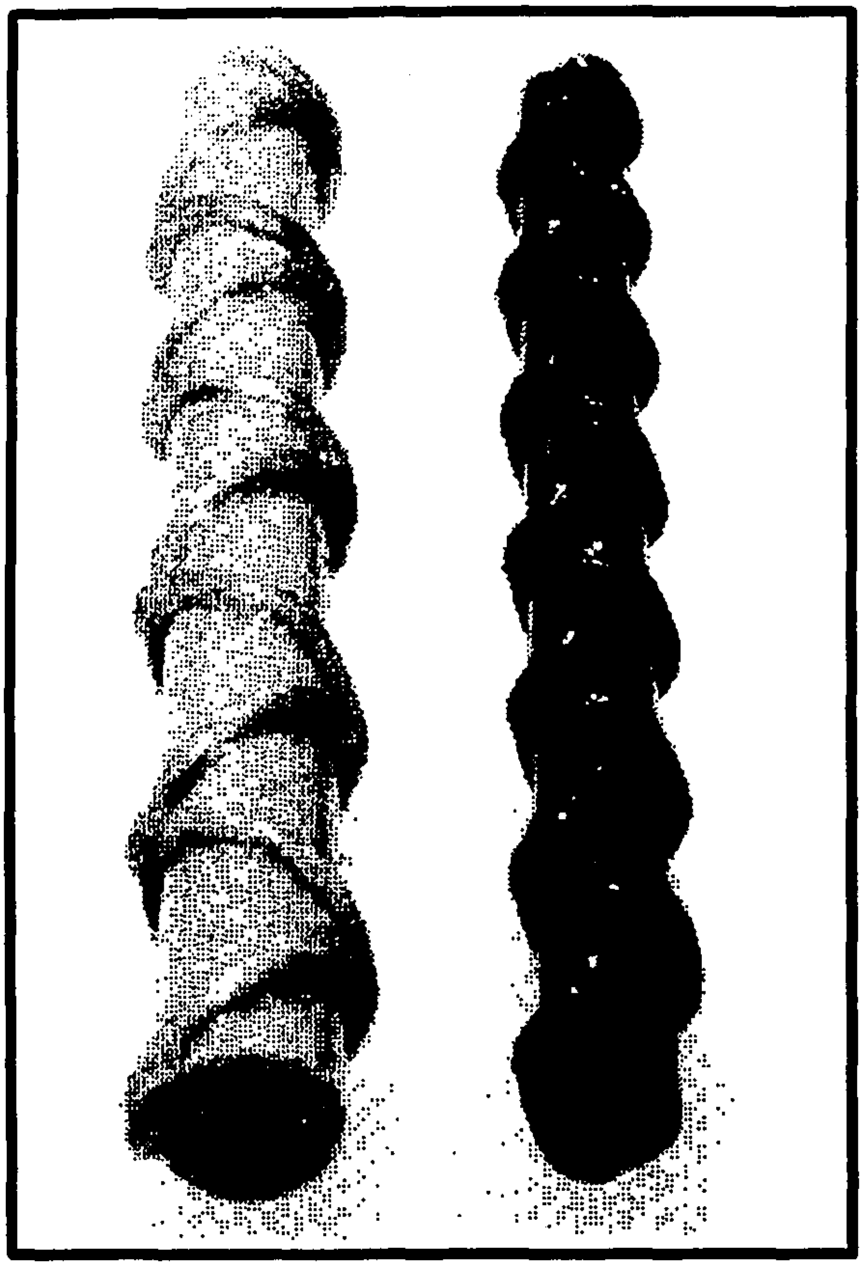


Figure 6.16: Isorod with helical surface profiles (Bedard, 1992)

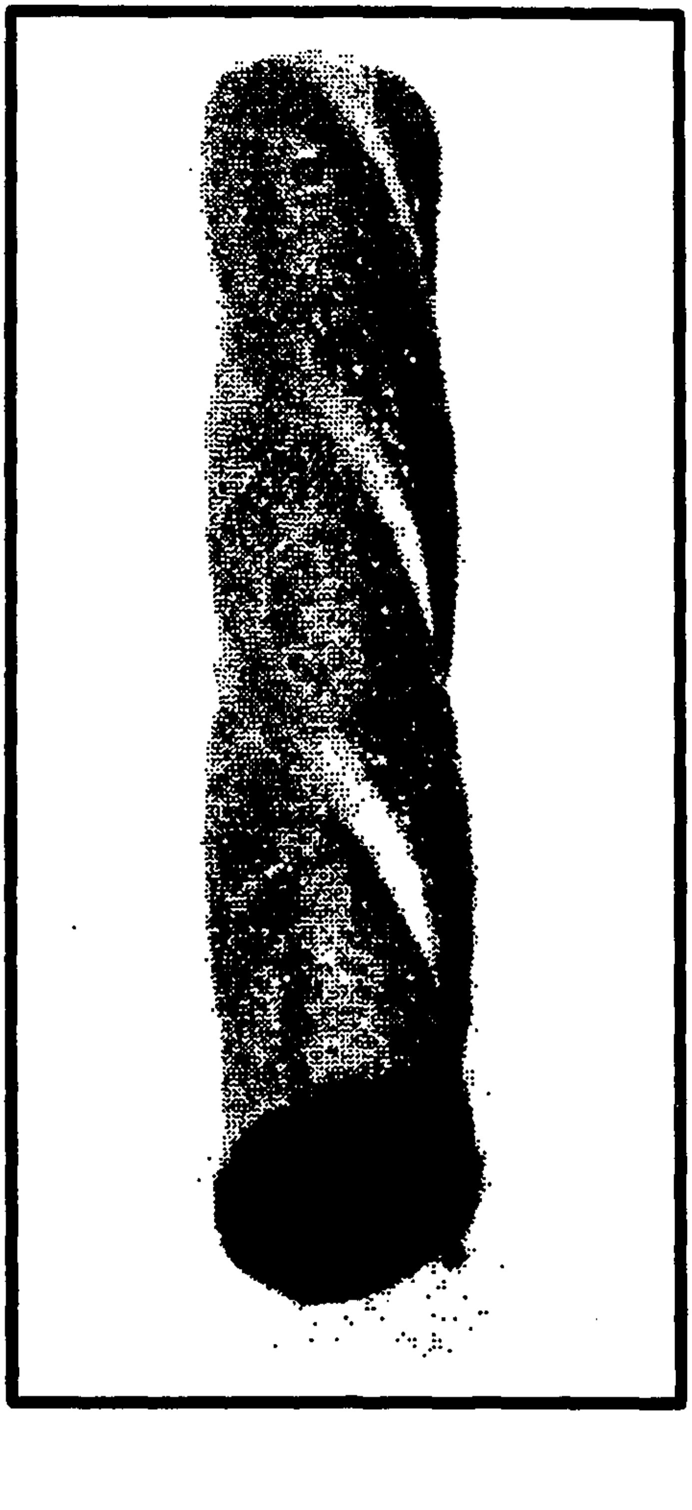


Figure 6.17: PSI Fiberbar with deformed surface profile (Bedard, 1992)

6.9 Thiessen Fiberglass Rockbolt

Thiessen rockbolts are also made up of glass fibers and have a deformed surface profile i.e. a band of glass fibers spirally wound around the bolts. These bolts are presently used for rock reinforcement in ore passes by Inco Ltd., in their Manitoba Division.

6.10 Kevlar

Introduction

Kevlar is a new type of fibre, introduced by DuPont (1971). Kevlar fibers are high strength (more than 3500 MPa) and high modulus fibers, being chemically and mechanically stable over a wide range of temperature. Kevlar ropes have been successfully used in marine applications because of its high strength and light weight. Kevlar is more expansive than other fibers and its surface is also very smooth.

Disadvantages

The common disadvantage (or problem yet to be solved) with all the continuous fibre structure and pultruded composites is the unalterable surface profile. Until now composites have been used only in aerospace and sporting goods application; and a smooth surface profile was developed which is compatible with such applications. For the mining environment, a rough and crude (sanded) surface profile is necessary to yield successful results. No attempts have been made to produce rough surface cable-bolts for mining. Moreover, poor bonding between ground and composite cables and insufficient end-anchorage discourages its use in mining. A thorough research program exclusively for mining applications should address these issues.

Kevlar products are very expensive and all the manufacturing processes are designed to produce the limited types of components used in the aerospace and other sophisticated applications. Cable-bolts capable of being used in mining structures need a modified manufacturing process which can produce less expensive mining compatible cable bolts. Kevlar can only be used as flexible cable-bolts not as rigid rock-bolts.

Applications

Kevlar 49 aramid fibre is designed primarily to be used as a reinforcement in epoxies, polyesters and other resins for higher performance applications, including aerospace and military design, sporting goods, boat hulls, flywheels, cables and tension members. Kevlar 29 aramid fibre is used mainly for ropes and cables and for protective clothing.

Ropes and Tension Members

Ropes and tension members have been used as deep sea mooring lines, mining and drilling rigs, antenna guys, oceanographic equipment and yacht rigging. Kevlar 29 and Kevlar 49 fibers are flexible resin-coated fibers are quickly replacing steel. With the highest strength-to-density ratios of any materials known, kevlar cables offer increased payloads and permit easier handling with smaller, lighter and more economical equipment. In addition, kevlar cables are cuttable and a potential use in continuous hardrock mining is obvious, provided the existing anchoring problem is solved.

Applications in Mining

Since Kevlar ropes have been successfully used as tension members in submarine applications (light weight and high resistance against corrosion), successful application should be anticipated in mining.

A high strength and low or high modulus cable with rough surface profile would be deemed to be necessary. Kevlar cables can be produced from 5 mm diameter to more than 250 mm diameter. With similar installation procedures, different patterns (birdcage, single cable with spring steel anchor, double cable taped together at various intervals, two cables with steel or plastic spacer or buttons, partial bridge) of Kevlar cables could be studied to obtain an optimum design for Kevlar cable-bolts. An adequate grout and grout injection pump for optimum cable design would also be a prerequisite for an efficient Kevlar cable-bolt system.

Kevlar cable can be produced to be more flexible than steel, which would open a new horizon in design-patterns of cable like continuous and U-shaped cables. Kevlar is a light weight material; this makes installation and transportation simple and more economical. Kevlar is cuttable, so this would fulfill the basic requirement for a tunnel support system in continuous hardrock mining.

Apart from cable applications, Kevlar sheet could be used to extinguish fire. Due to its high resistance to almost all chemicals, Kevlar cabins could be used to store and protect flammable and toxic chemicals.

6.11 Extren Fiberglass

Introduction

Extern fiberglass is a combination of fiberglass and thermoset polyester or vinyl ester resin. Extern fiberglass is high strength and lightweight. It is a dimensionally stable, corrosion resistant and anti-skid surface profile product. Extern is produced in numerous structural shapes like I-beam, L-beam, flat-plate, threaded nut and bolts, etc. Anti-skid surfaces have been used to solve the problem of poor binding and lack of friction resistance in tensile load bearing members (Figure 6.18). Extern can be used as rockbolts only. It is much more expensive than steel products. The cost of 25.4 mm diameter solid rod is C $ 16.27 per meter.

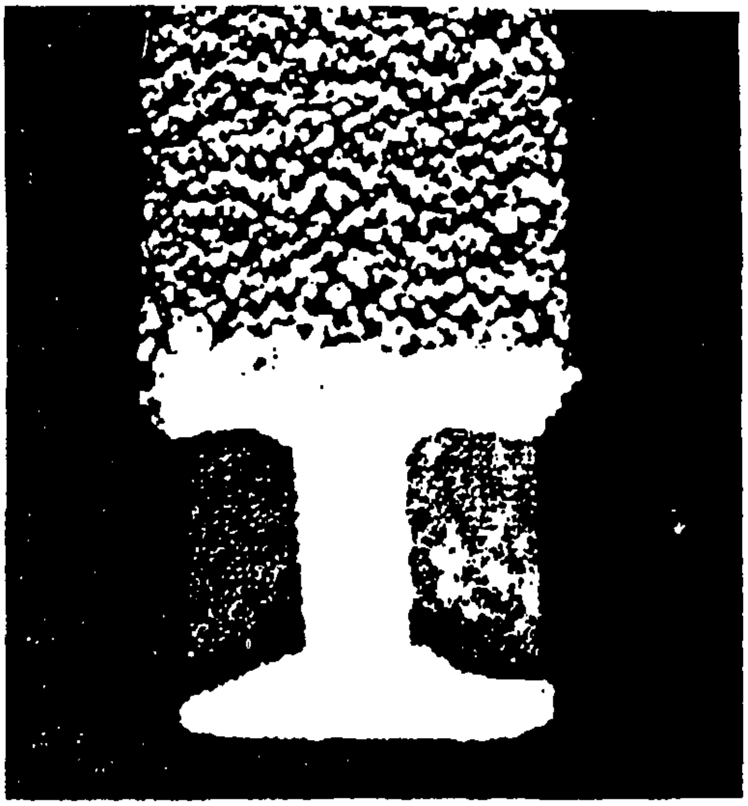


Figure 6.18: Anti-Skid surface of an I-beam (Extern Fiberglass Technical Brochure)

Advantages

The following are the advantages of Extren fiberglass bolts/tendons which enables it to replace steel and aluminum.

- High strength.
- Light weight - 80 % less than steel.
- Dimensional stability.
- Corrosion resistance.
- Non-Conductive; thermally and electrically.
- Low maintenance.
- Surface profiles are alterable.
- Anti-Skid bolts/rod are possible.

Disadvantages

The use of Extren fiberglass in underground mining would appear to be limited to rock-bolts only. Because of its stiffness, Extren cannot be used as cables. Extren fiberglass is relatively expensive compared with other types of fiberglass, which is another hindrance to any use in mining.

Application in Mining

As already mentioned, Extren fiberglass can be used as rock-bolts in all types of underground mining environments. In addition, Extren is produced in numerous structural shapes like I-Beam, L-Beam, flat-plates, threaded nut and bolts etc. So whenever there are problems associated with chemicals, corrosion, dampness and fire, Extren structures could be used. Extren has already been installed as large platforms in chlorine manufacturing plants in Texas.

Mechanical Properties

Table 6.12 shows the mechanical properties of Extren circular rods.

Table 6.12: Properties of Extren Rods and Bars (Extern Fiberglass Technical Brochure)

Property	Test Method (ASTM)	Longitudinal
Tensile Strength, MPa	D638	689.5
E-Modulus, GPa	D638	41.37
Compressive Strength, MPa	D695	413.7
Flexural Strength, MPa	D790	689.5
Density kg/m^3	D792	1993.14

6.12 Newport Composites

Introduction

Newport composites (NC) are manufactured through a braiding process and end products are in the shape of rigid or flexible tubular tendons. One of the main benefits of tubular braid relates to its use in fabricated tubular structures and its inherent torsional strength and stiffness. Biaxial braid is the most commonly used form of braided material due primarily to its inherent ability to conform to various shapes and diameters, thus yielding a versatile material for tubular applications. By introducing third fiber orientation parallel to the braid axis (at the time of braiding) a triaxial braid is created providing added bending/torsion strength and stiffness to the curved parts. With the addition of resin, tubular braid tendons of varying stiffness can be obtained.

Advantages

High strength, low or high elasticity modulus (can be modified), chemical and corrosion resistance, and light weight are the properties of NC tubular tendons. Cement or chemical grout can be injected from inside and outside of the hollow NC tendon, which can give a high bonding strength. One of the advantages of braiding manufacturing process, which is adopted by Newport manufacturers, is vary the amount of braids in a tubular structure. 1 cm to 15 cm diameter braid form pre-impregnated roving of glass, aramid and carbon are available. Cost and strength of tubular tendons are directly proportional to each other. In applications where high strength tendons are not required, low cost and low strength tendons are the alternative to economize the structural support systems.

Disadvantages

NC is more expensive than several other composite materials and because of its unique flexible nature, the existing parameters of support systems cannot be applied to it. Moreover, existing installation and grouting techniques would have to be modified to install tubular flexible tendons.

Application in Mining

Newport flexible tubular tendons could be used as tensile members in underground mining in place of steel cables. These tubular tendons could be grouted from inside and outside. This would give a strong bond strength, high pressure resistance and high shear strength.

6.13 Ryton-PPS

Introduction

Ryton-PPS is a pultruded thermoplastic glass or carbon fibre composites. There are two types of products: one contains reinforcement in the form of a fiber mat specially designed for processing by a rapid compression molding process, often referred to as thermoplastic stamping. The other contains reinforcement in fabric form specially designed for laminated and thermoforming. Ryton-PPS has numerous structural shape products like solid and hollow pipes, flat and curved plates, molded shapes, etc. Ryton-PPS have been applied in various fields such as industrial equipment (e.g. housing and internal parts of chemical pumps, compressors, heat exchangers, valves, piping and ducting components etc.), aircraft components (their examples are seat backs, arm rests interior panels and doors, etc.), oil field equipment (e.g. submersible pumps, downhole drilling equipment etc.) and automotive (e.g. internal engine parts, underhood covers, etc.). Ryton-PPS tendons are very expensive compared to other composites. The price of 12. 7 mm diameter solid red is $ 30.0 per meter and 19 mm diameter solid rod is $ 57.5 per meter.

Advantages

Ryton-PPS has both solid and hollow shape tendons, which has the potential to replace steel rockbolts in mining applications. Different surface profiles, although expensive yet can be produced to increase the bonding between bolt and grout. Like other fiber glass products, the amount of glass fibers can be varied to achieve an optimum mechanical properties of Ryton-PPS products. The nominal mechanical properties of two types - Glass and Carbon are given in Table 6.13.

Disadvantages

Ryton-PPS is very expensive as compared with other available cuttable tendons. Like other glass and carbon reinforced composite products, the smooth surface profile of Ryton-PPS should be modified to obtain a high bond strength between grout and tendon.

Table 6.13: Mechanical properties of Ryton-PPS (Technical Brochure, Phillips)

Properties	Units	Reinforcement Type	
		Glass	Carbon
Density	g/cc	1.92	1.52
Fiber content volume	%	55	55
Tensile strength	kPa	790	5447
Tensile modulus	GPa	41.37	117.21
Elongation	%	2.0	1.1
Flexural strength	kPa	965.3	1172.15
Flexural modulus	GPa	34.47	96.53
Compressive strength	kPa	689.5	896.35
Compressive modulus	GPa	47.37	103.42

6.14 Fiberloc

Introduction

Fiberloc polymer composites belong to the family of glass fibre reinforced materials. Fiberloc compounds are based on specially tailored vinyl matrices and utilize patented glass coupling technology to optimize the properties such as strength and stiffness, creep and fatigue resistance, low coefficients of expansion and dimensional stability. At the same time, these properties are obtained without sacrificing most of the traditional properties of Poly Vinyl Chloride (PVC) such as chemical resistance, processability, appearance, flame retardancy and economic viability. Final products are numerous shapes of injection molding materials and extrusion materials like automobile engine parts, solid and hollow pipes, curved and flat plates etc. The cost of 25.4 mm diameter solid rod is C $ 60.0 per meter

Applications

The following are application areas of Fiberloc Polymer composites:

Construction

Shelves, I-beam, ceiling grids, drywall components, cooling towers, sheds, posts, fencing, panels etc.

Plumbing

Pipe couplers, chlorinators, pump housings, valve components and end caps.

Automotive

Cartop racks, sheet, accessories and trim.

Marine Products

Ladders, railings, navigational housings and trim.

Advantages

Fiberloc products have high chemical resistance with excellent mechanical properties. Therefore, they have multiple applications like rockbolts, cable bolts, different parts of conveyor belts, storage tanks, tools, etc.

Disadvantages

Fiberloc is very expensive. Its surface profile is not very smooth and can be compared with the rough surface of wooden dowels. An intensive and costly research program would be required to produce design guidelines.

Mechanical Properties

Table 6.14: shows the nominal mechanical properties of Injection Molding Materials (IMM) and Extrusion Materials (EM):

Table 6.14: Mechanical Properties of Injection Molding Materials (IMM) and Extrusion Materials (EM) (Technical Brochure, BFGoodrich, 1986)

Properties	IMM	EM
Glass Content wt. %	20	20
Tensile Strength, MPa	88.25	63.43
Tensile Modulus, GPa	6.62	5.10
Elongation %	2.7	2.4
Flexural Strength, MPa	155.14	101.35
Flexural Modulus, GPa	6.69	5.07
Coefficient of Thermal 10^{-5}in/in $^{\circ}$F	1.3	1.8
Melting Temperature $^{\circ}$C	196 - 204	188 - 199

6.15 Mincon Rockbolt

Mincon rockbolts are made up of glass fibers with pultrusion manufacturing process and have a deformed surface profile i.e. a band of glass fibers spirally wound around the bolts. Their technical information data is not available.

6.16 Discussion

This chapter summarizes the concept of composite material products as support systems in underground and surface mining. A brief description of raw materials, commonly used in tensile bearing elements, are presented. The manufacturing processes exclusively used for tendons are also highlighted.

In a commercial survey, more than sixteen different types of patented composite tendons are identified, which have the significant potential to be applied as rock bolts and/or cable bolts in underground mining. Some of the products such as Arapree, Weldgrip and Weidmann were exclusively produced for mining or civil engineering applications. Therefore, these products have compatible parameter to the existing ground supports. However, there are other composite products, which at the present conditions, can not be used as rock supports. Nevertheless, they have potential to be modified and applied as tensile bearing elements. Ideas such as tubular tendons, which can be grouted from inside and outside to increase their strength, are discussed herein. These ideas are at their infancy at the present time, however they can be implemented in near future. These can result in solving complicated ground support problems and can be available as an economical alternative in future. Table 6.15 summarizes the mechanical properties of the patented composite tendons, discussed in this chapter.

Tale 6.15: Mechanical properties of patented composite tendons

Tendon type	Nominal dia. (mm)	Tensile strength (MPa)	Tensile modulus (GPa)	Strain at failure (%)	Density (kg/m^3)
Arapree	20 x 5*	1330	125	2.24	1250
Weldgrip	20	955	na!	na	na
Weidmann (type KO)	22	568	na	na	na
Polystal (type V6000U)	21.7	816	na	na	na
Polyglas	22	210	17.2	na	1716
Celtite	26	140 - 700	14 - 40	2	na
Kodiak	24.5	690	41 - 50	na	2282
PSI Fiberbar	22	1034	41.4	na	na
Isorod	22	700	44.8	na	na
Kevlar	na	na	na	na	na
Extren	22	690	41.37		1993
Newport	na	na	na	na	na
Ryton PPS (glass)	22	790	41.37	2.6	na
Fiberloc (IMM)	24.5	88.25	6.62	2.7	na
Thiessen	22	194	na	na	na
Mincon	22	na	na	na	na

* rectangular shape; ! not available

Chapter 7

Preliminary Laboratory Investigation on Composite Tendons

7.1 Introduction

This chapter presents the results of pull out tests of the commercially available composite tendons, identified and discussed in previous chapter. The tests considered the effect of grout-bolt interface on the overall support behaviour. On the basis of the findings of this chapter, a thorough investigation of acceptable tendons were conducted and their results are presented in forthcoming chapters. The rest of the composite tendons, although not suitable due to many reasons such as their smooth surface, high cost and potentially hazardous loose fibers, yet, not rejected completely. These tendons can be used in special applications in geomechanics.

7.2 Composite Tendons

Sixteen different types of composite tendons were selected from a survey of approximately one hundred composite manufacturing companies. Some of these tendons were produced exclusively for mining applications such as Weidmann and Weldgrip. While some for civil engineering applications such as Arapree, etc., however, these tendons can also be applied in mining applications, for example, flexible composite cables might be used to pre-reinforce the stopes. Some of the tendons were produced only for uniaxial tensile load bearing applications without having a specific use. Each and every tendon type has its own unique physical and mechanical properties. The cross-sectional area and shape, end-conditions and surface profiles are different from each other. Some of the tendons have special coating on their outer surfaces such as Polystal. They posed different types of problem during experiments then tendons without special coating. Likewise, some tendons have proportionally more matrix material than the fibers and so they have different response during pull out experiments. Longitudinal and radial stiffness of the tendons also has an effect on their response during pull out process.

Before starting experiments, all these parameters were addressed. It was thought to adopt the same pull out test setup, which was used for steel cables. A standard chuck (cable gripping device) was available for steel cables. But, no such gripping device was available for composite tendons. Most of experiments which were conducted by the commercial industry (see such test results in chapter 6, for example) to find out the tensile strength of composite tendons, were normally done on individual "fibers". These results were based on net cross-section of the fibers. Their results, therefore, can not be generalized for mining applications. The tensile strength of composite tendons can not be summed up from tensile strength of individual fibers alone, but it depends on other variables such as the proportion of matrix material and its mechanical and physical properties, orientation and length of fibers, different types manufacturing defects, etc. Therefore, it was deemed necessary to investigate thoroughly the composite tendons in mining constraints.

7.2.1 Anchoring Devices

Various attempts were made to design and develop anchoring devices in the laboratory which could be used for all types of tendons. In the trials, tendons were gripped from both ends with the help of chucks and then these were pulled away in R.D.P. Howden testing machine (detail about testing machine is given in Section 4.3.1). Different anchoring devices were devised and tested including conventional grips of universal testing machine.

No hydraulic gripping systems were either available at the Rock Mechanics Laboratory of the Department of Mining and Metallurgical Engineering or the Structural Laboratory of the Department of Civil Engineering and Applied Mechanics. The following problems were encountered during the testing of gripping devices with composite bolts:

- Two types of conventional grips were available to hold the bolt in the specified position. One with two flat steel wedges and the other with four conical shape steel wedges. Flat wedge grip is tightened before commencing the experiments while four wedge grip gets tightened with the application of axial load on the bolt. From the experiments, it was found that it was impossible to obtain an optimum amount of tightening with these conventional grips. If the grips were tightened to strictly hold the bolt without allowing any slip, then the wedges crushed the bolt locally. On the other hand, if the grips were kept loose to avoid crushing of the bolts, then bolt slipped at about 20 - 30 percent of ultimate tendon breaking load.
- Most of the conventional gripping systems are made of steel and they are not compatible with the composite materials. Due to difference of stiffness, steel wedges tend to crush the bolts.
- The two-wedge grip is 3 inch long while four wedge grip is 1.5 inch long. In order to increase gripping area and length, six inches long steel grip was developed but it also failed in its performance since it partially crushed the bolt and consequently bolt slipped out around 40 percent of tendons breaking load.
- Another problem, which was observed during the experiments, was non-uniform distribution of load exerted from the grip on the bolts. This induced excessive load on few fibers and no or very little load on other fibers. As a result, bolt was sheared with some 20 - 30 percent fibers detached and pulled out from the bolt. The observed load was approximately 30 % of breaking load of the tendon.

- Keeping all the above factors in view, a request was made to Akzo (Gerritse, 1987) to get their gripping system. The description of gripping system is given in Section 6.1. This system contains an outer steel casing and two inner polyamide wedges. Polyamide is a chemical material and its mechanical properties are compatible to Arapree, therefore it worked properly. Some slippage was observed in the beginning of pull out tests of Arapree with polyamide grip, however, this minor slippage stopped on tightening of the gripping mechanism. This slippage was measured and later on deducted from the graphical representation of pull out data. No such facilities were available at laboratory to produce chemical grip compatible to glass fiber tendons. Besides, this was behind the scope of this research program to produce such gripping devices.

7.3 Laboratory Test Setup

Due to unavailability of a universal gripping system which could have allowed the testing of different types of composite tendons with similar variables and facilitate comparison of their pull out performance. A technique was devised to achieve both purposes; to pull out the grouted bolts without inducing "unknown" slip and to obtain the results which could be compared. This technique or method is shown in Figure 7.1. It consists of two steel pipes, the longer pipe to hold the bolt is termed as the anchoring pipe or anchoring length and the shorter pipe, to test the bolt-grout interface; is termed the testing pipe or testing length. Anchoring pipe was kept more than twice the length of the testing pipe. This pipe arrangement was adjusted in a specially design frame (as shown in Picture 7.1). In the pull out process, steel pipes were pulled away while keeping anchoring pipe frame fixed to the base of the R.D.P testing machine and testing pipe was allowed to move upward. Additional LVDTs were mounted on both free ends of tendons to measure their displacement. This testing arrangements made sure to fail the testing portion only and, therefore, no slippage was occurred in the anchoring length. Same data acquisition system, which is given in Section 4.3.1, was used to obtain the load and corresponding displacement.

A steel pipe with 63 mm inner diameter was used to model the host medium for tendons which have a diameter of approximately 20 mm. For smaller diameter tendons i.e. 3 mm to 7 mm diameter, a steel pipe with an inner diameter of 40 mm was used. The same constant rate of loading was used, which was used for steel cables i.e. 1 mm/minute.

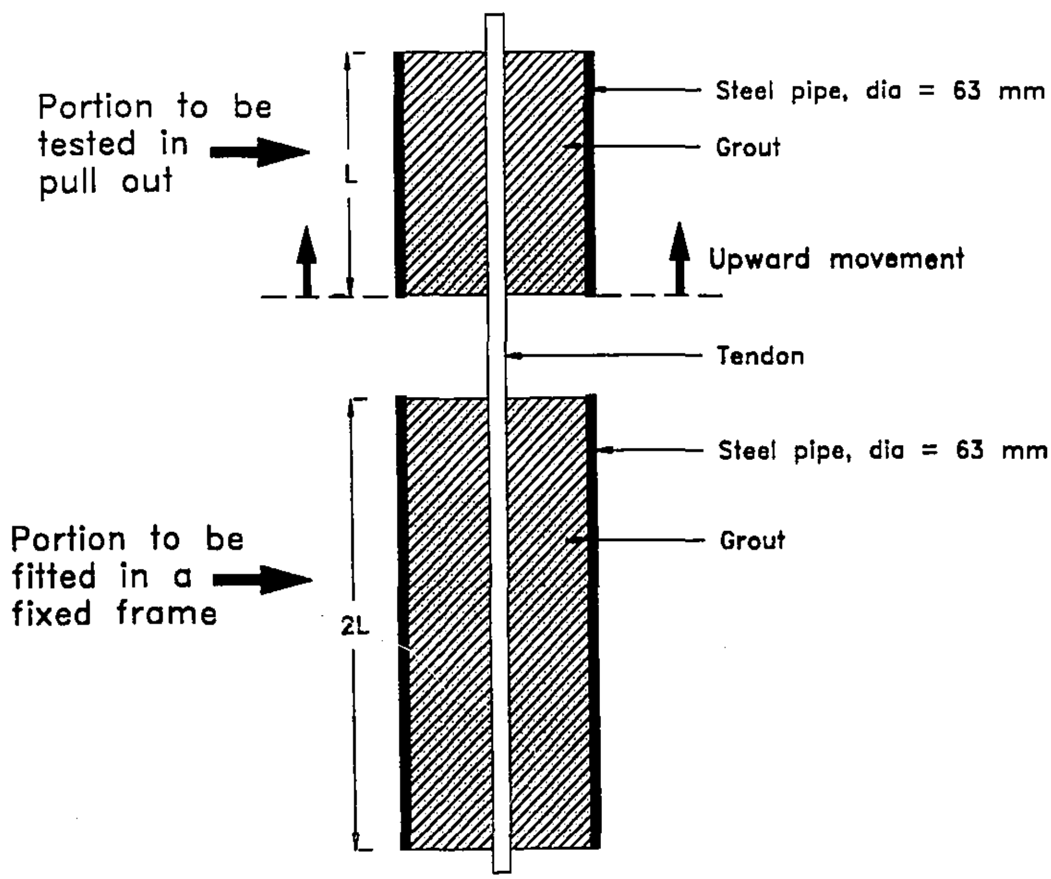
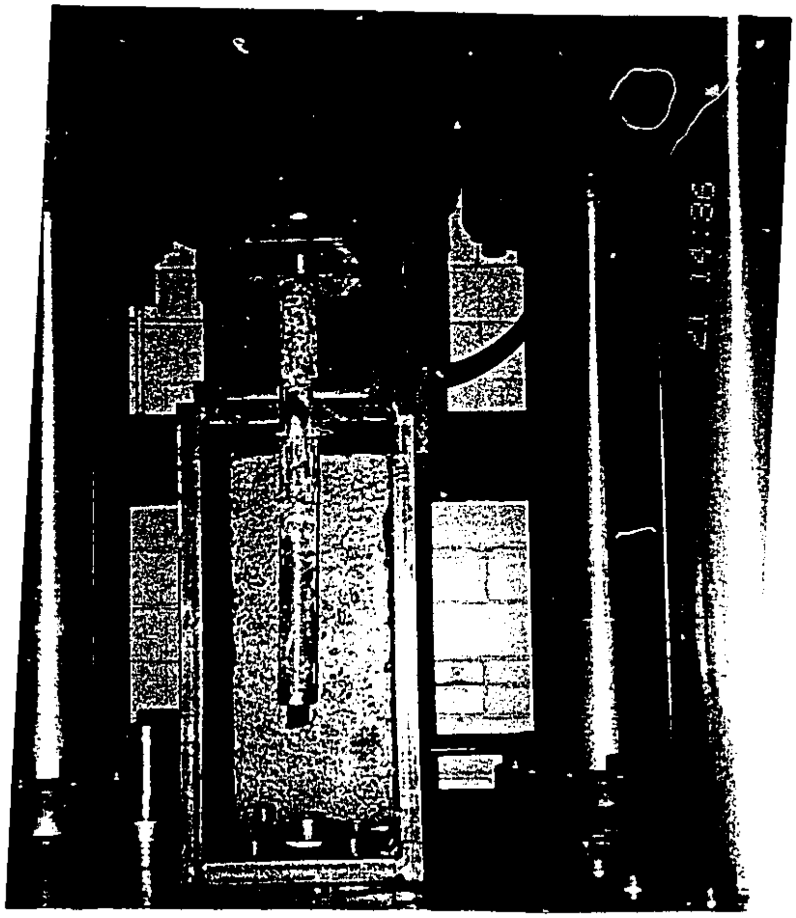


Figure 7.1: Two-steel pipe pull out technique



Picture 7.1: Two-steel pipe pull out experiment in pull out frames

7.4 Experiment Parameters

The purpose of experiments at this preliminary stage was to evaluate the performance of different types of composite bolts embedded in grout. To achieve this purpose, a relatively small i.e. approximately 100 mm long embedment length was selected for the pull out tests. According to Littlejohn (1992), the distribution of shear stress at tendon-grout interface under maximum load (i.e. tendon-grout interface failure load) remains constant for small lengths, as shown in Figure 7.2. For this preliminary stage of experiments, a firm basis from such a technique could be obtained to asses and compare the performance of composite tendons with conventional bolts.

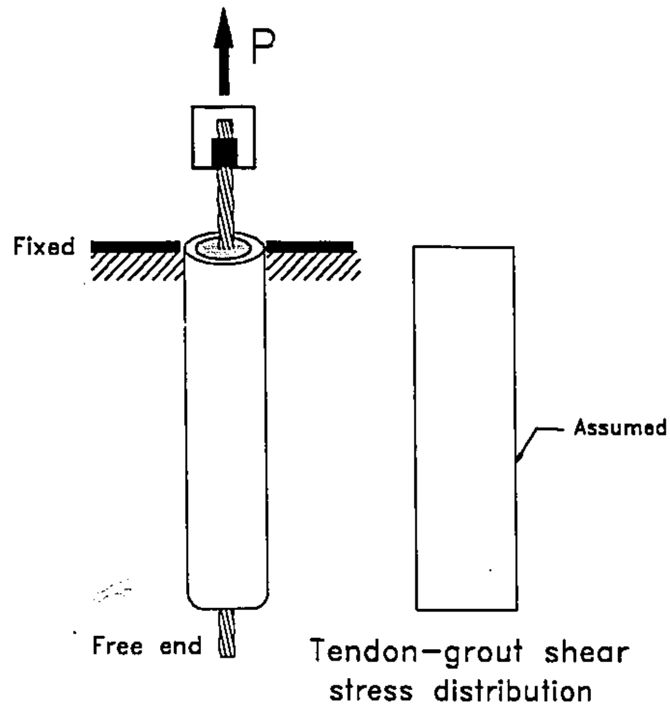


Figure 7.2: Assumed shear stress distribution at tendon-grout interface

7.5 Experiment Test Results

The following is the description of pull out tests conducted on various types of composite bolts. A brief discussion on each tendon type is also mentioned along with the results. All efforts were made to keep parameters constant to obtain a realistic comparison.

7.5.1 Conventional Steel Cable

Although comprehensive experimental evaluation of fully grouted steel cable parameters are given in chapter 4, but for the purpose of comparison, steel cables were tested again with the same experimental technique, explained earlier, as was adopted for other composite tendons. The pull out test results of three samples are given in Figure 7.3. From this figure, it can be seen that due to small embedment length, the stiffness of pull out system reduces, however at the same time, unexpectedly, the load carrying capacity increases significantly. This increase can be associated with this specific two-steel pipe testing technique. Moreover, due to the relatively small embedment length, the cable started to slip under the axial load before cracks could generate in grout and could reduce its radial (confining) strength, and as a result, grout remained intact and uncracked. When the cable-grout interface reached to maximum stress (τ_{max}), cable started slipping without shedding any load. Additionally, frictional forces increased against the slipping resistance, because no cracks were formed due to smaller embedment length, and thus increased the pull out capacity. It is also seen from the figure that cable started to "slip and stick" in the post elastic region. It is worth while to mention that no rotation of cable was observed during the pull out process.

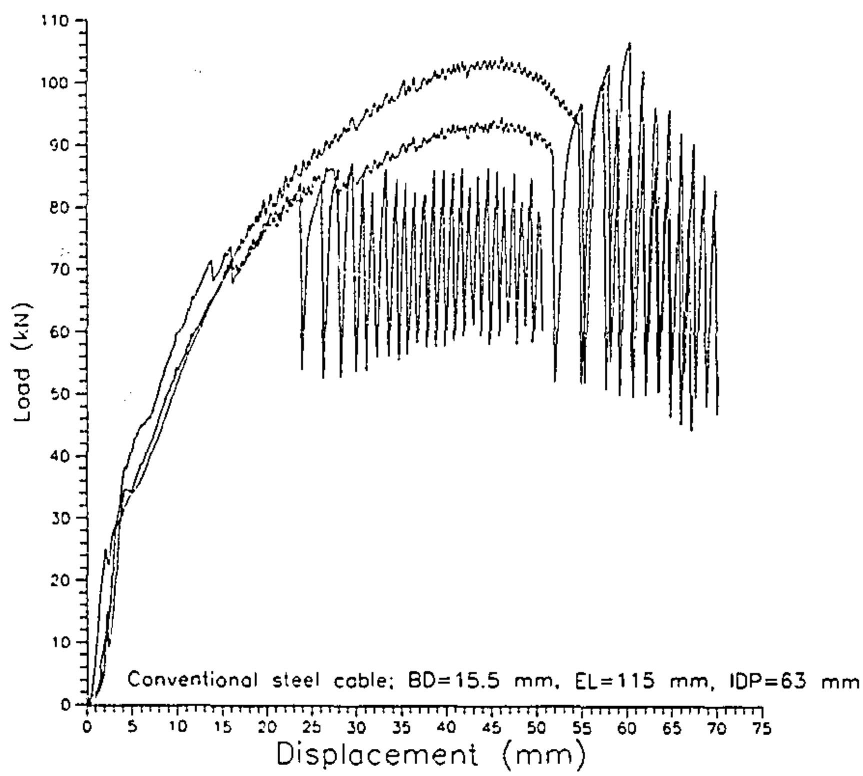


Figure 7.3: Conventional steel cable, tested in two-steel pipe technique

7.5.2 Weldgrip Rockbolt

Weldgrip is a rigid rockbolt with small knobs on the surface. Both hollow and solid bolts of 22 mm diameter are commercially available. Both types were embedded in cement grout and after 28 days of curing, bolts were pulled out. Their pull out response is more or less identical to the pull out response of conventional steel rebars (see Figures 7.4, 7.5 and 7.19). The peak load carrying capacity of hollow Weldgrip bolts is lower than the solid bolts. The other difference, which is more significant, is the residual load carrying capacity of both bolts. The solid Weldgrips give this capacity between 20 - 30 kN whereas hollow bolts do not resist pull out force in this zone. This difference is most likely due to radial (or transverse) stiffness of the bolts. This preliminary pull out test performance proved that Weldgrip bolts have potential to be used in place of steel rockbolts. Therefore, Weldgrip rockbolts are selected for further detailed investigation.

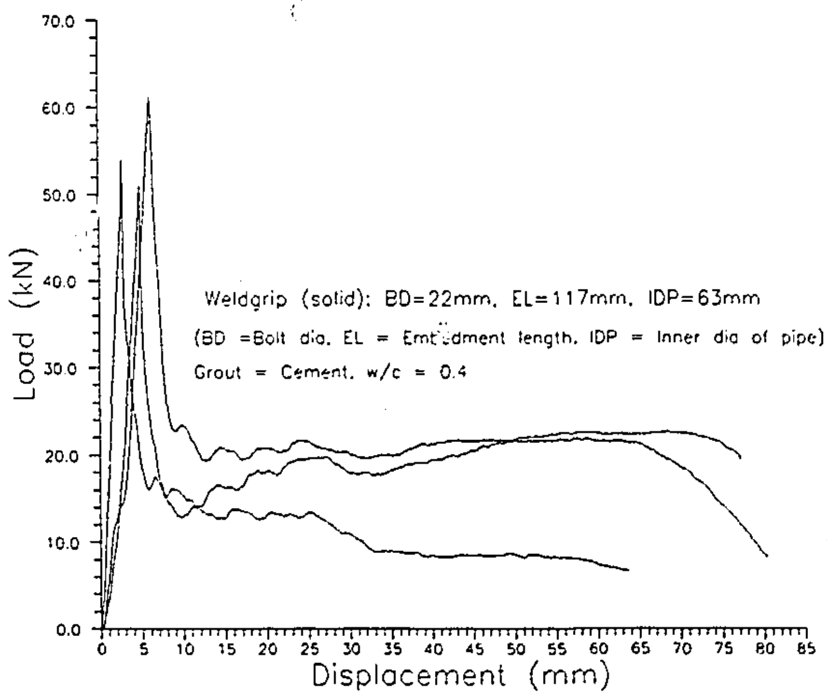


Figure 7.4: Pull out capacity of Weldgrip (solid) rockbolts

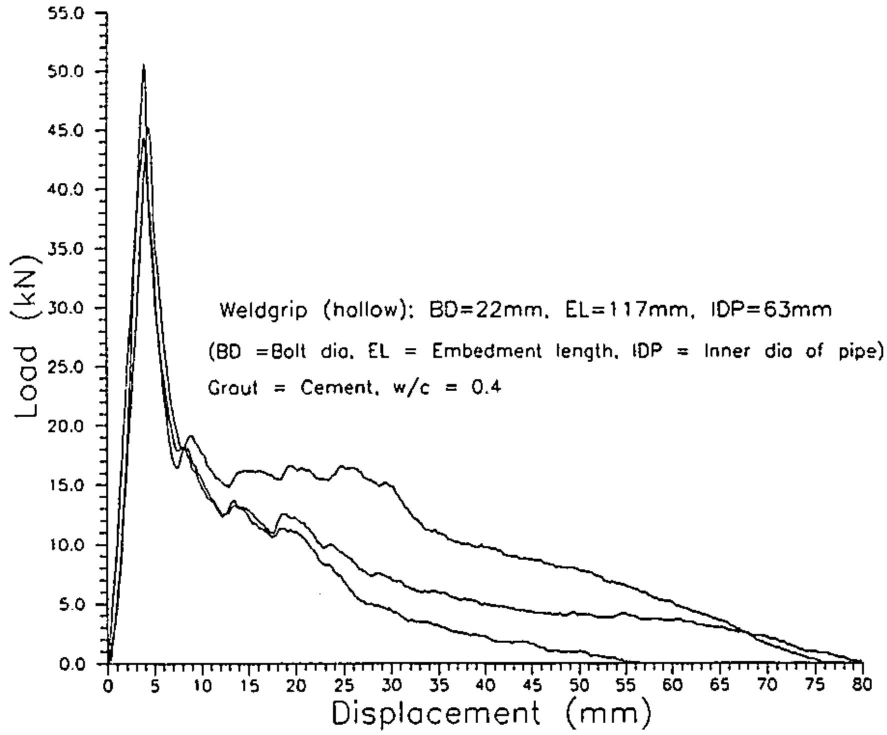
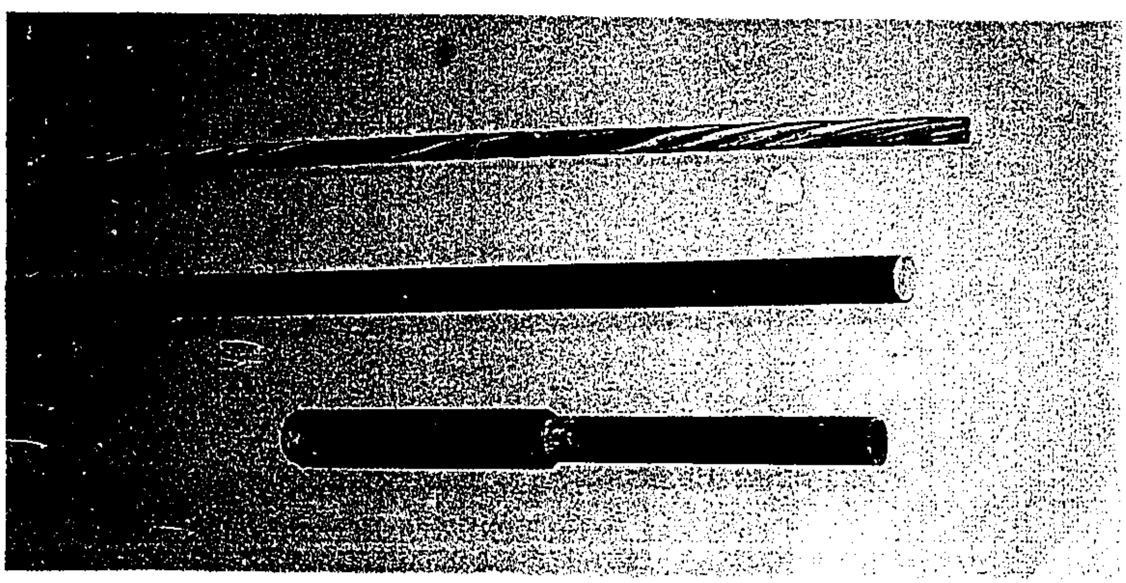


Figure 7.5: Pull out capacity of Weldgrip (hollow) rockbolts



Picture 7.2: Photograph of steel cable (top), solid Weldgrip (middle) and hollow Weldgrip bolts (bottom)

7.5.2 Arapree Flexible Tendon

Arapree is a flexible tendon made up of continuous aramid fibers. Two types - rectangular and circular Arapree tendons were pulled out in order to study their tendon-grout interface behaviour. Figure 7.6 reveals that rectangular Arapree tendon has peak load close to 30 kN while no or little residual capacity. On the other hand, Figure 7.7 represents the pull out response of 8 mm diameter round Arapree tendon, which shows that maximum pull out capacity was close to 20 kN and residual load carrying capacity 7 kN. Because of this performance, Arapree tendon was also selected for further research.

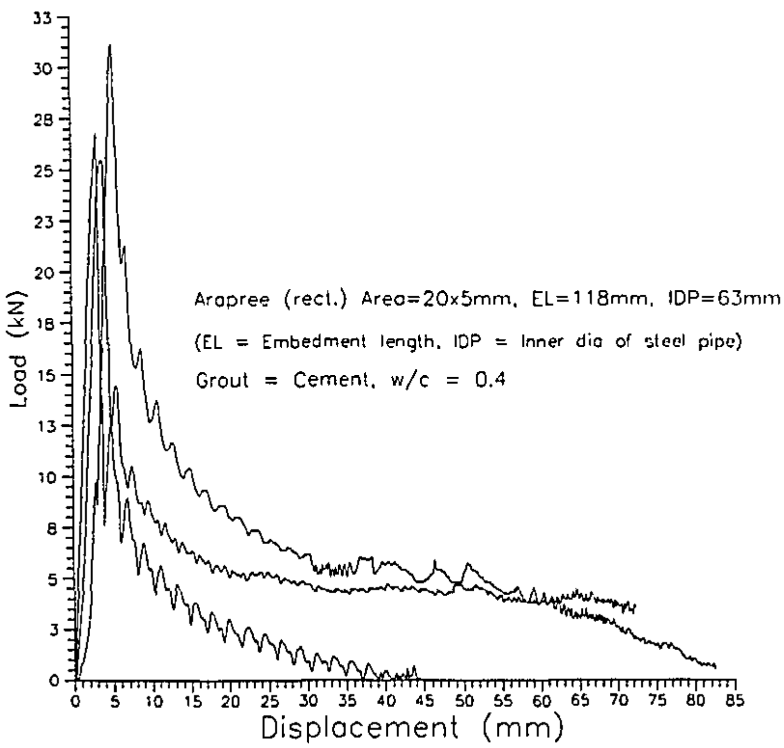


Figure 7.6: Pull out response of rectangular Arapree

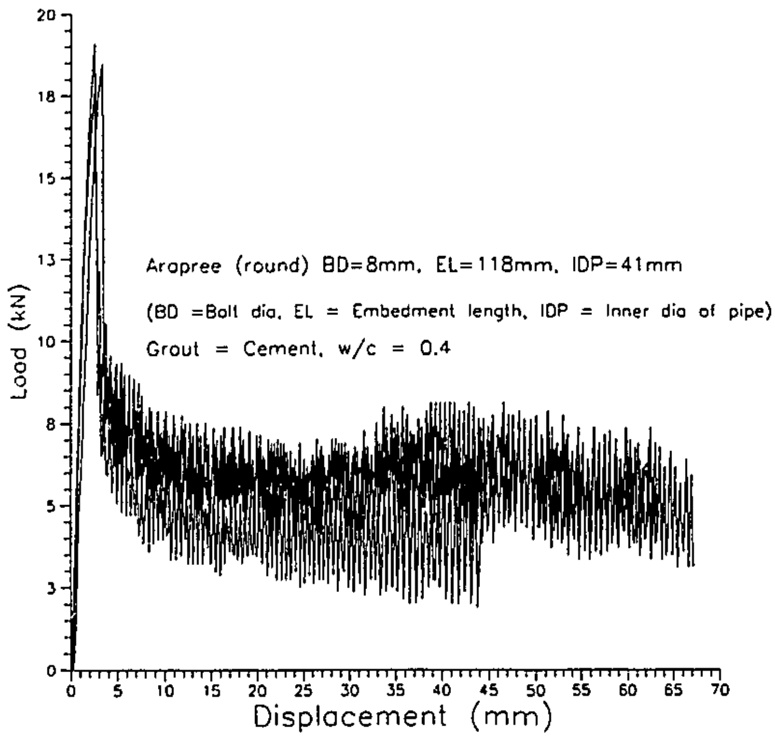
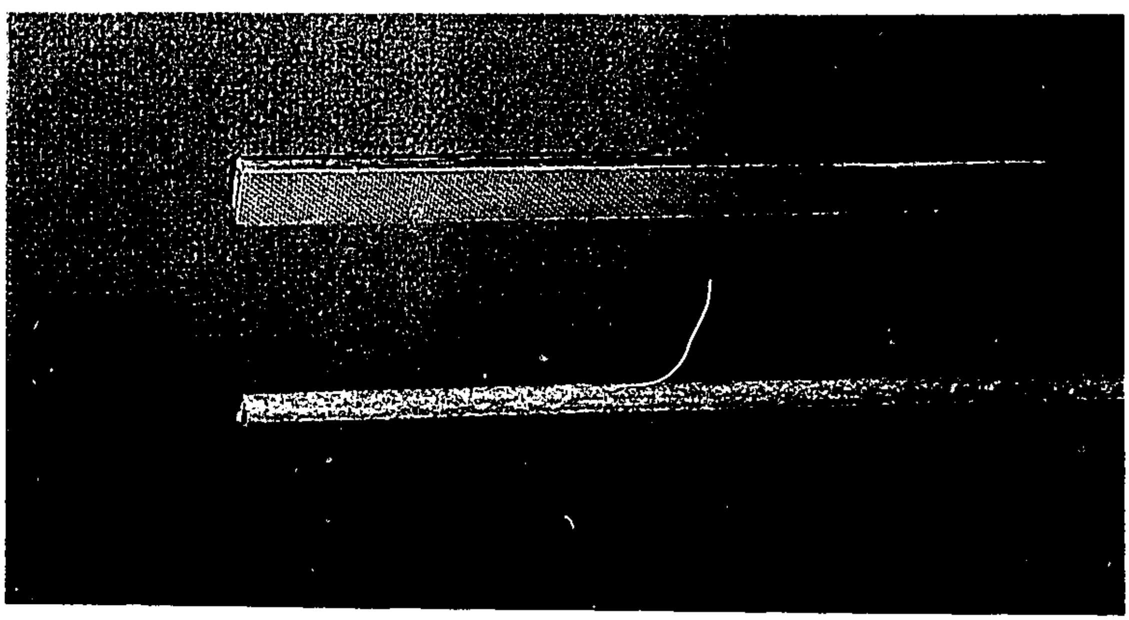


Figure 7.7: Pull out response of round Arapree



Picture 7.3: Photograph of rectangular Arapree (top) and circular Arapree (bottom) tendons

7.5.3 Celtite Rockbolt

A peak load of approximately 20 kN and residual load carrying capacity of 12 kN was obtained in the Celtite rockbolts pull out experiments (see Figure 7.8). It was found, during the experiments, that these bolts have loose fibers in the hull, therefore, they can not be recommended for mining application. As a matter of fact, loose fibers created a rough surface which increased the bonding between grout and tendon. Further modifications are required such as coating of bolts with special sealants to conceal the loose fibers and prevent their loose ends to expose to air and moisture. At the same time, a rough surface profile has to be created to improve the bonding between bolt and grout. This bolt was not selected for further detailed laboratory investigation.

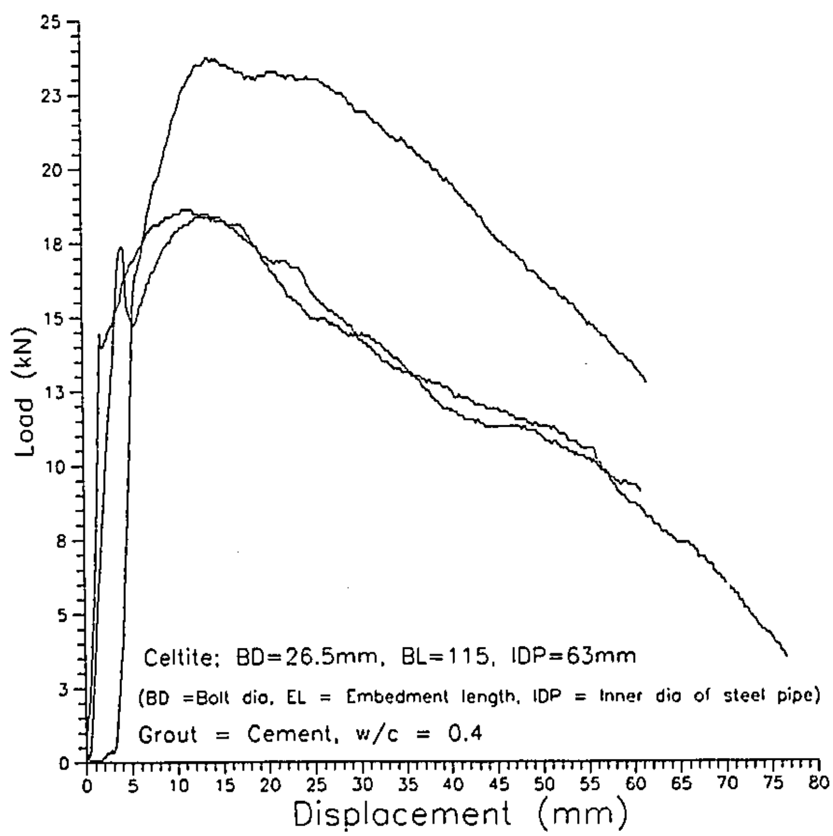
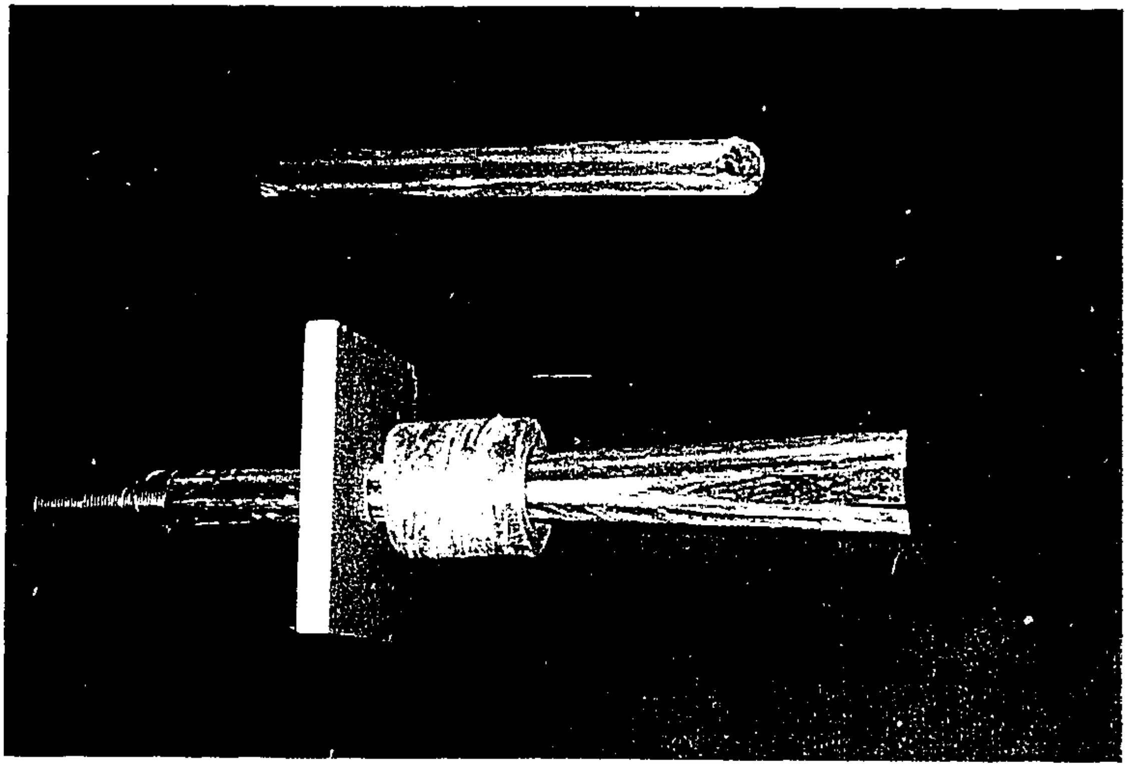


Figure 7.8: Pull out behaviour of Celtite rockbolts



Picture 7.4: Photograph of Celtite rockbolts

7.5.4 Weidmann Rockbolts

Weidmann rockbolts were developed for exclusive use in coal mine in place of wooden dowels. Therefore, their standard of comparison is the wooden dowels rather than steel bolts. Their prime purpose was to compact the loose coal ground by inserting them into it so that ground should not collapse during and after excavation. Weidmann rockbolts have a very smooth surface, therefore a 22 mm diameter bolt only gives 15 kN of peak load and 5 kN of residual load (see Figure 7.9). Moreover, they are very expensive and not a viable alternative to existing rock bolts. Hence, these are not considered for further research.

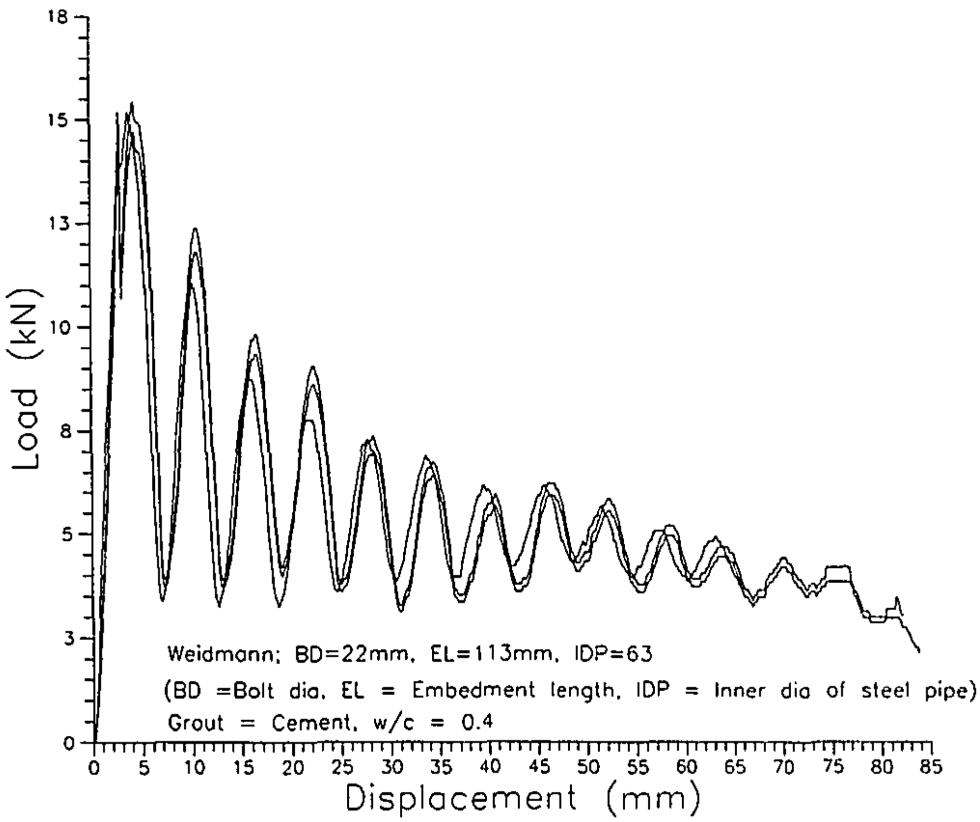
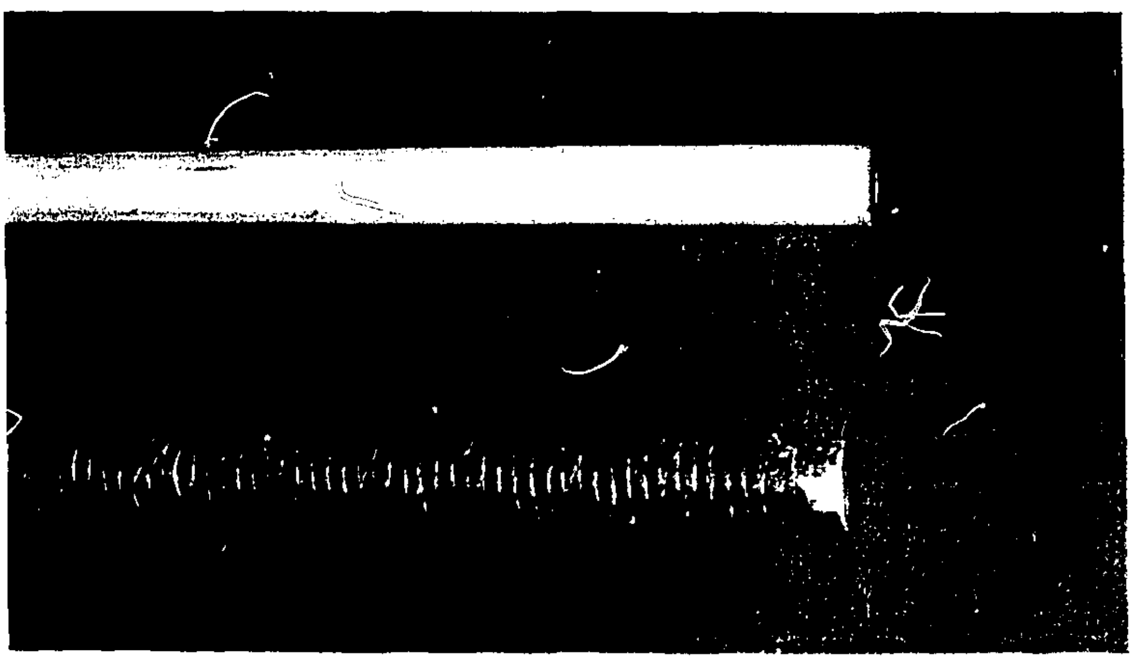


Figure 7.9: Pull out behaviour of solid Weidmann rockbolts



Picture 7.5: Photograph of Weidmann rockbolts

7.5.5 Polystal bolt

Polystal is a fiberglass product. It is almost as flexible as steel cable, but its diameter is normally 7 to 8 mm. Polystal tendons with and without cover are commercially available for tensile load carrying purposes. Due to their smooth surface profile, the bonding between Polystal tendons and grout is relatively week, as shown in Figure 7.10 and 7.11. Their overall behaviour closely resembles the general pull out behaviour of conventional bolts. The covered Polystal offers more resistance to pull out force than Polystal tendons without cover. However, the former failed due to delamination of outer cover. Both types of tendons, after pull out tests, showed no failure of fibers and physically they remained intact. The pull out behaviour can only be improved by converting Polystal cable into birdcage formation (Mah et al. 1992).

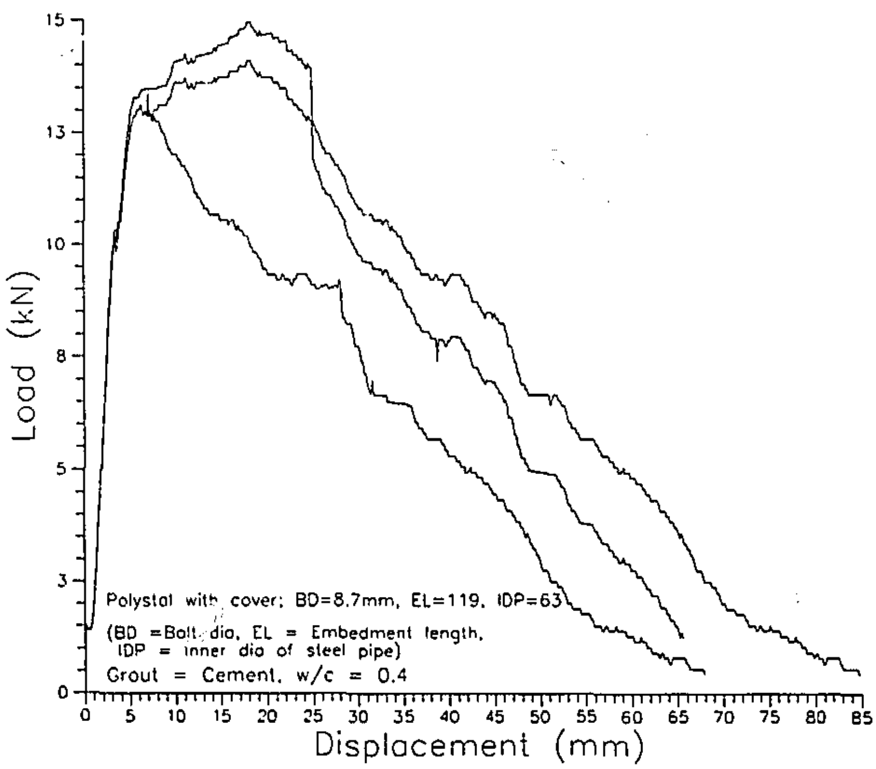


Figure 7.10: Pull out behaviour of Polystal with cover

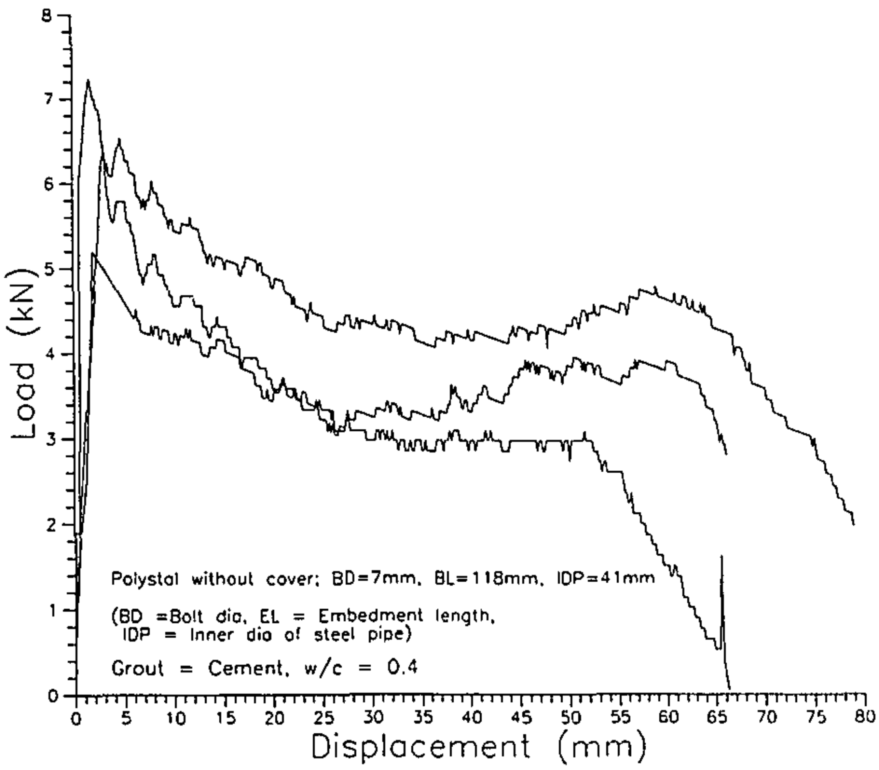
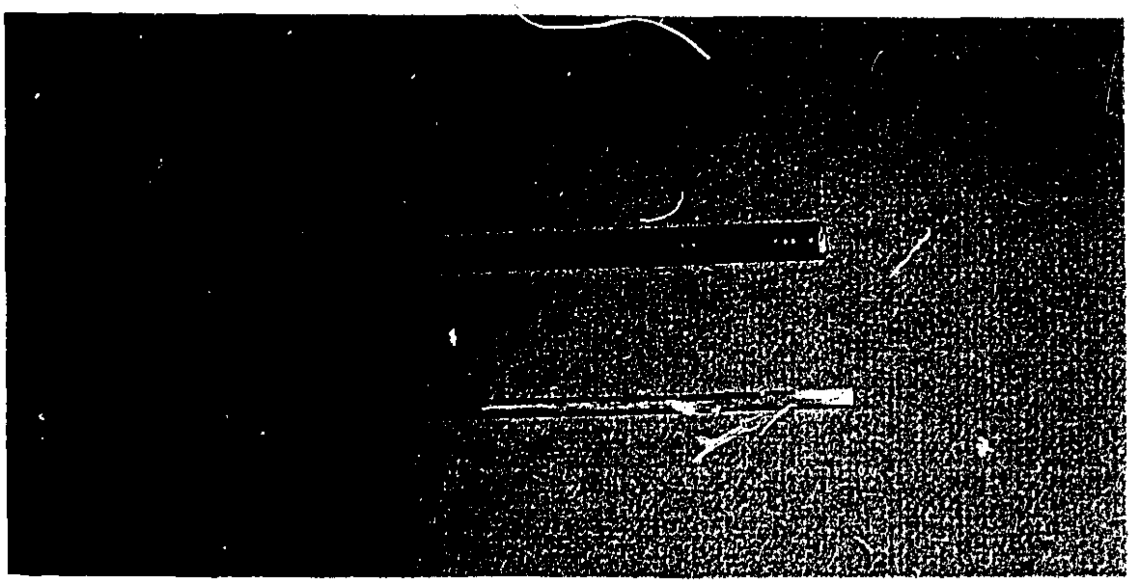


Figure 7.11: Pull out behaviour of Polystal without cover



Picture 7.6: Photograph of Polystal with cover (top) and Polystal without cover (bottom)

7.5.6 Thiessen Rockbolt

Thiessen rockbolts are also product of glass fibers. These bolts have spiraled grooves on their outer surface. This makes outer surface very rough and, as a result, gives a very strong bonding between grout and bolt, as shown in Figure 7.12. The pull out pattern of Thiessen rockbolts and conventional bolts are similar. The major disadvantage of Thiessen rockbolts is that its behaviour is too brittle, as can be seen from Figure 7.12, that the bolt broke physically at about 110 kN. Therefore, it cannot be recommended for mining applications. However, if the tensile strength of bolts is increased, then these can be used to reinforce such rock structures which need not to be relaxed after reinforcement. Thiessen rockbolts, due to their insufficient tensile strength and ductility, are not accepted for further research.

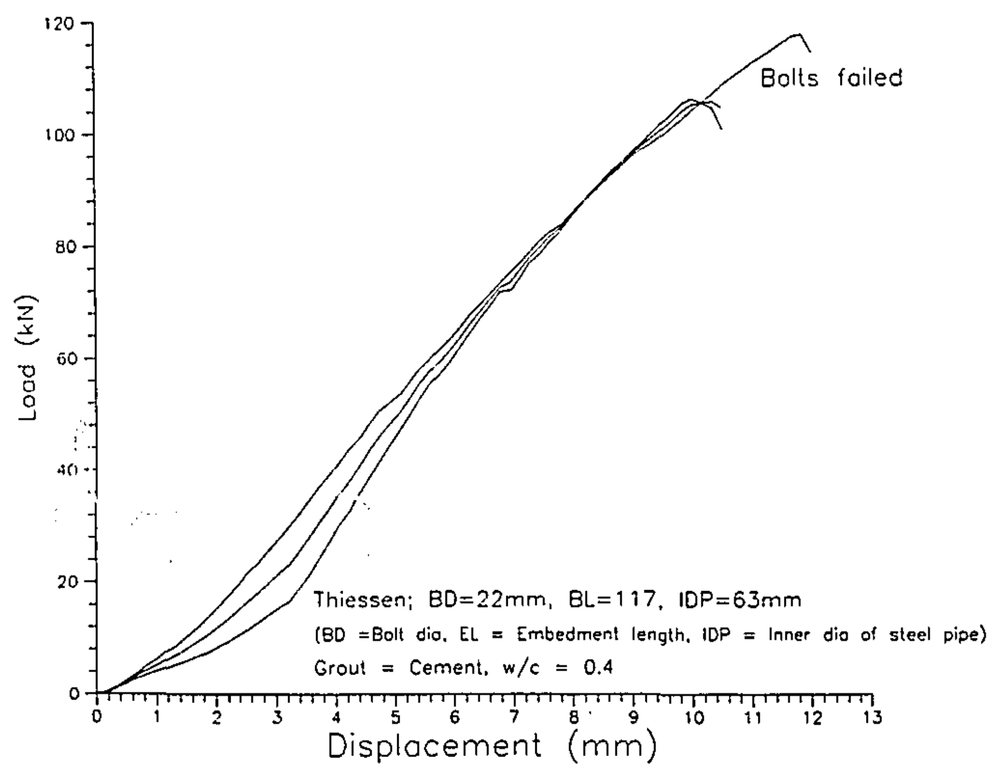


Figure 7.12: Pull out behaviour of Thiessen rockbolt

Picture 7.7: Photograph of Thiessen rockbolt

7.5.7 Mincon Rockbolts

Mincon rockbolts are also fiberglass product and their nominal diameter is 22 mm. A rough surface profile has been created by gluing bands (thin ropes) of glass fibers, in a spiral shape, on the outer surface of the bolts. This operation of creating rough surface is carried out after the manufacturing of bolts. The failure of bolt grout interface initiated with the delamination of outer rough surface. Figure 7.13 shows pull out behaviour of Mincon rockbolts. Only the elastic portion is identical to conventional bolt pull out tests, whereas, post-peak behaviour is unique. It is believed that this unique behaviour is due to the spiraled surface profile. Mincon rockbolts were not selected for further research because of two reasons; (1) these bolts also have loose fibers on the outer surface, which caused great irritation in the skin during the experiments, and (2) as long as bonding between glued glass bands and the glass fiber tendon has not been improved, the failure will always be initiated through this debonding and pull out response would not be improved.

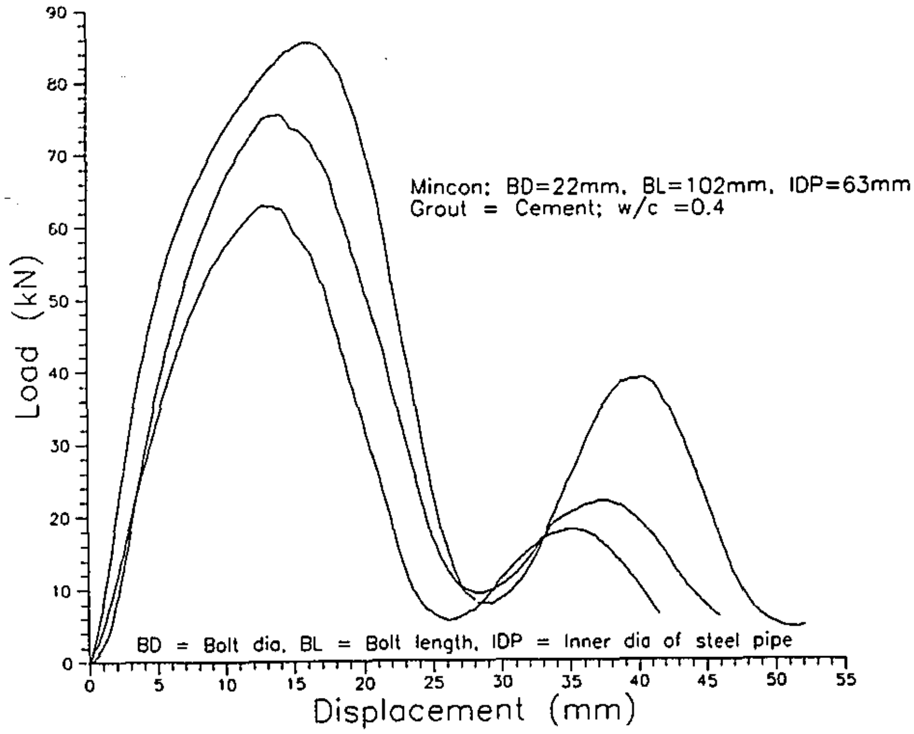


Figure 7.13: Pull out behaviour of Mincon rockbolt

Picture 7.8: Photograph of Mincon rockbolt

7.5.8 Remaining Composite Tendons

All those composite tendons whose pull out capacity came out to be less than 10 kN were not accepted for further research. The basic reason for the low pull out load is their smooth (slippery) surface profile. The pull out response of these tendons is shown in Figure 7.14. Newport flexible and hollow tendons were grouted inside and outside, but they failed due to breakage of tendons. No slippage was observed during the pull out process. Polyglas slipped out of the grout because of its very smooth surface profile. E-glass tendons are yet not commercially available, nonetheless, these were tested to evaluate the bonding between E-glass fibers and grout. The commercial patented product is not available.

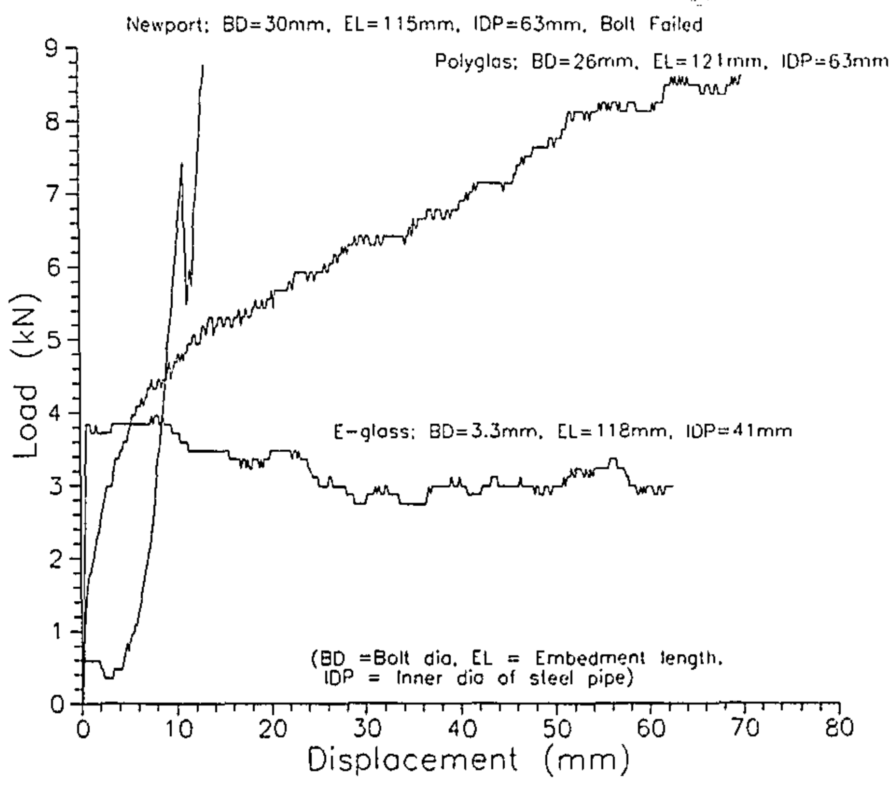


Figure 7.14: Composite tendons; whose pull out capacity is less than 10 kN

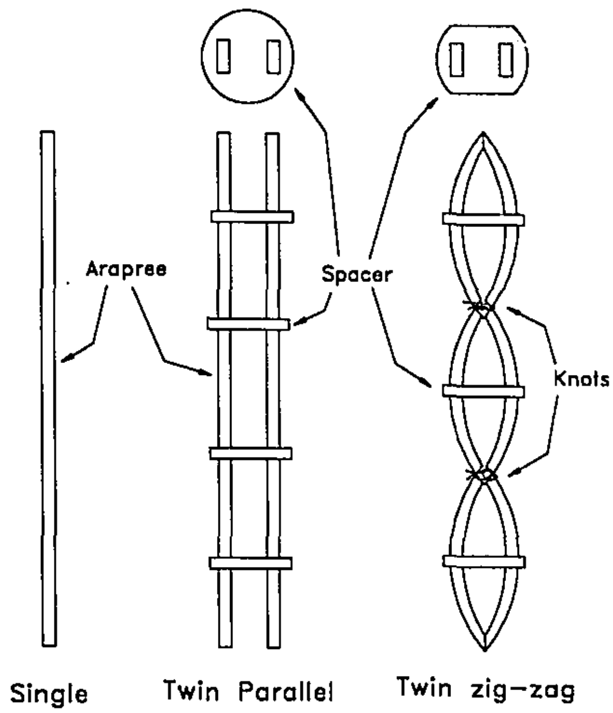
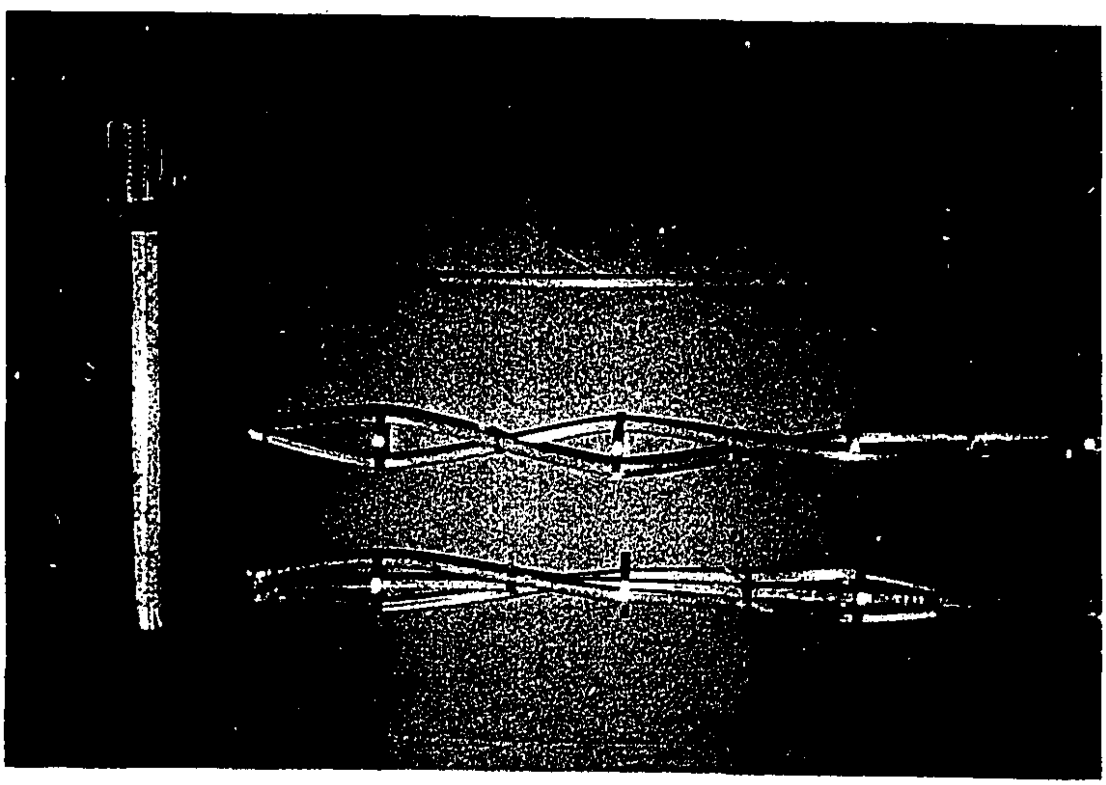


Figure 8.2: Various configurations of Arapree tendons



Picture 8.1: Different configuration of Arapree tendons

8.3.1 Single Arapree Tendon

There are five types of commercially available Arapree tendons; see Table 8.1. The table also shows the mean breaking strength of tendons and corresponding dimensions. All the available types of Arapree tendons both round and rectangular shapes were investigated in the mining constraints. The pull out tests of all types of Arapree are graphically illustrated in Figure 8.3 and their test parameters are given in Tables 8.2 and 8.3.

From this figure the following conclusions can be drawn:

1. All those Arapree tendons types whose load bearing capacity is less than 6 tonnes with embedment length of 300 mm (12 inch) are not considered suitable for mining applications.
2. The round sanded shape Arapree pulled due to the delamination of outer skin. It is observed that the inner tendon remains unaffected i.e. no failure of any fiber was seen. This type of surface profile is not suitable in the tensile load bearing mining applications because neither does it give the critical bond strength nor adequate failure warning.
3. The observed pull out failure of rectangular Arapree f400000 was due to the debonding of the Arapree-grout interface. The post-failure behaviour shows that the residual load carrying capacity is approximately 30 kN.

Table 8.1: Different types and shapes of Arapree

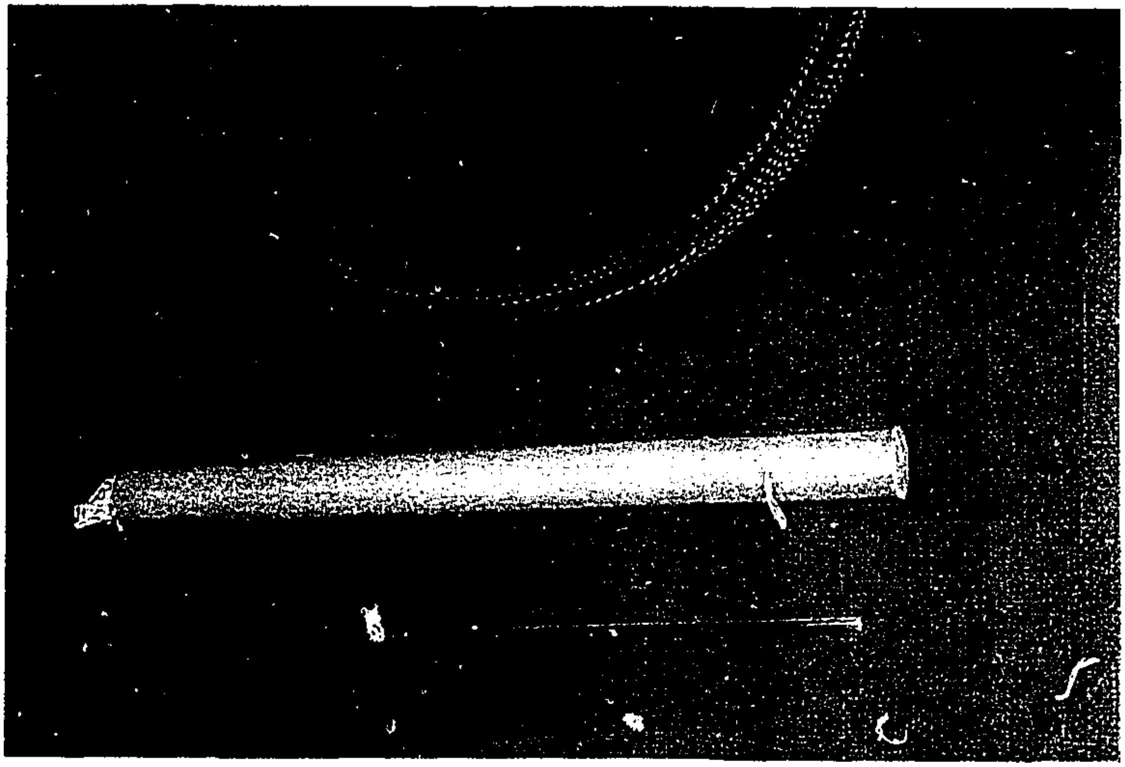
Arapree Type*	Dimension (mm)	x-sectional Area (mm^2)	Mean Breaking. Force (kN)
f 100,000 (RS)	3.9	22.2	33.3
f 200,000 (RL)	7.9	22.2	66.6
f 100,000 (SS)	1.5 x 20	11.1	33.3
f 200,000 (SM)	2.6 x 20	22.2	66.6
f 400,000 (SL)	5.0 x 20	44.4	133.2

* see the abbreviations under the Table 8.2.

Table 8.2: Test parameters of different types of single Arapree

Sample Type*	Arapree Type	Arapree Dimension (mm)	$P_{ultimate}$ (kN)	$P_{residual}$ (kN)
ANC-SL	f400000	5 x 20	64 ± 5.2	22 ± 4
ANC-SM	f200000	2.6 x 20	53 ± 4.8	18 ± 5
ANC-SS	f100000	1.5 x 20	26 ± 2.1	TB
ANC-RL	f200000	7.9	27 ± 4.3	14 ± 1.8
ANC-RS	f100000	3.9	15 ± 2.2	TB

* A = Arapree; NC = Normal Cement (i.e. conventional cement); SL = Strip Large; SM = Strip Medium; SS = Strip Small; RL = Round Large; RS = Round Small; TB = Tendon broke



Picture 7.9: Photograph of Newport (top), Polyglas (middle) and E-glass (bottom) tendons

7.6 Discussion

The basic mechanical properties of all the above mentioned patented composite tendons are available in the commercial literature. These values, however, depend on the net cross-section of the fibers rather then the tendon itself. Figure 7.15 shows graphically specific tensile strength versus specific tensile modulus of various types of fibers. In order to evaluate the performance of tendons, pull out tests were conducted in the laboratory of the McGill Rock Mechanics Research Group. Since the chucks (or grips) were not available for all types of tendons, a technique as shown in Figure 7.2, has been developed. A relatively small embedded length was used so that shear strength developed at tendon-grout interface should remain uniform throughout the embedded length. An embedment length of approximately 110 mm was selected. Results of pull out testing of fourteen different types of tendons are shown graphically in Figure 7.16, 7.17 and 7.18. In order to have an overall comparison, pull out behaviour of conventional steel cable is presented in Figure 7.16 and test results of all the conventional bolts are shown in Figure 7.19 (Stillborg, 1986). A direct comparison of these bolts is not possible because tendon variables such as diameter and surface roughness are not constant. A direct comparison is most likely possible by comparing the shear strength of the tendon-grout interface. The following equation (Littlejohn, 1992) is used to calculate the shear stress (τ) at tendon-grout interface:

τ_{peak} = Maximum pull force / πDL

Where, D and L are the diameter and length of tendon, respectively.

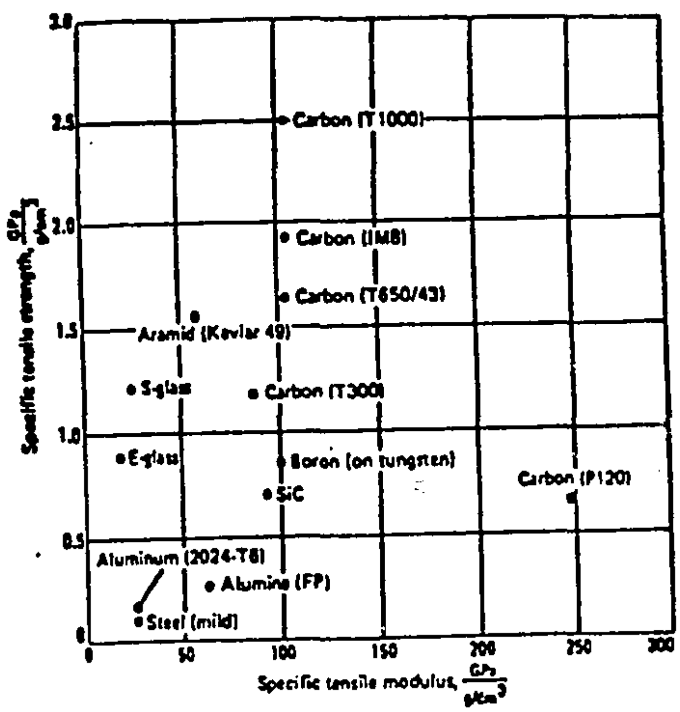


Figure 7.15: Comparison of different fibers (Reinhard & Clement, 1987)

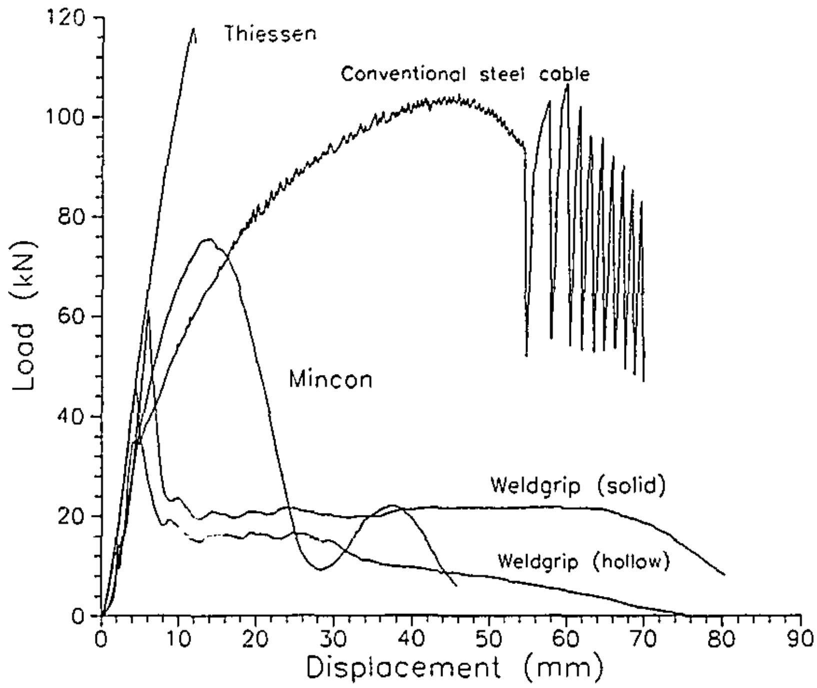


Figure 7.16: Pull out behaviour of composite tendons (when pull out capacity > 40 kN)

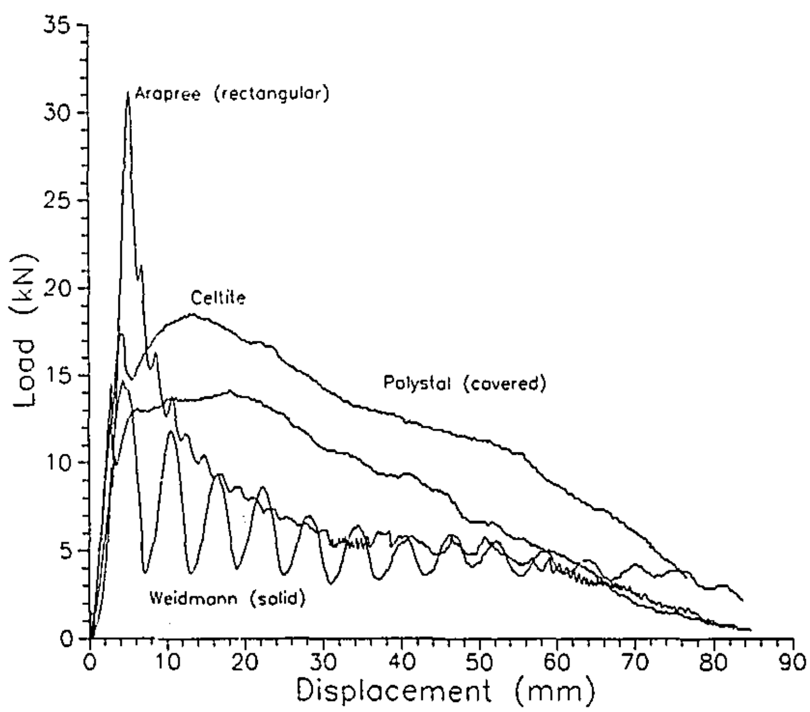


Figure 7.17: Pull out behaviour of composite tendons (when pull out capacity > 15 kN)

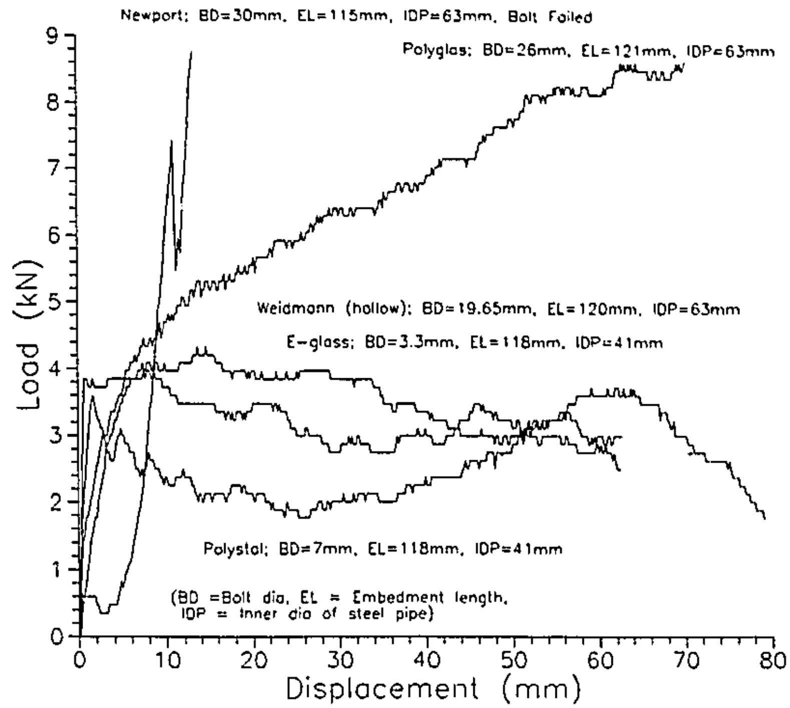


Figure 7.18: Pull out behaviour of composite tendons (when pull out capacity < 10 kN) .

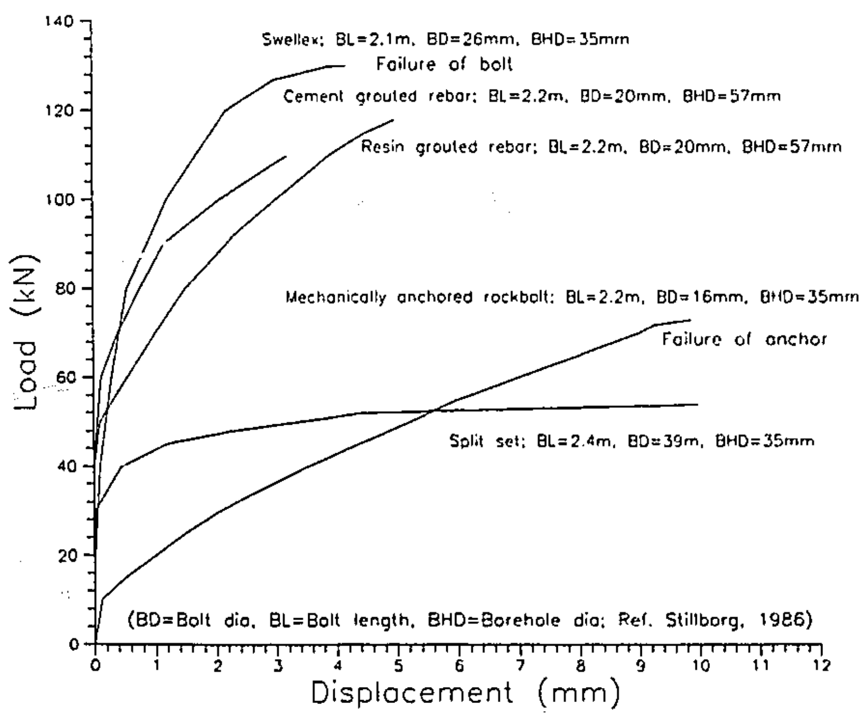


Figure 7.19: Pull out behaviour of conventional tendons (Stillborg, 1986)

Figure 7.20 shows the average maximum tendon-grout interface shear strength and Table 7.1 shows various experimental parameters as well as the interface shear strength of composite tendons.

The application of tendons, either flexible like cable or rigid like steel rebars, have been used mainly as reinforcing elements in a wide range of rocks such as coal, potash, sandstone and granite. There are large variation in deformational behaviour of rock mass surrounding an excavation. This is mainly due to different degree of inherent discontinuities as well as stress regime. Therefore reinforcement of rocks should take into account these important variables. Choquet [1991], after surveying support in thirteen Quebec mines wrote:

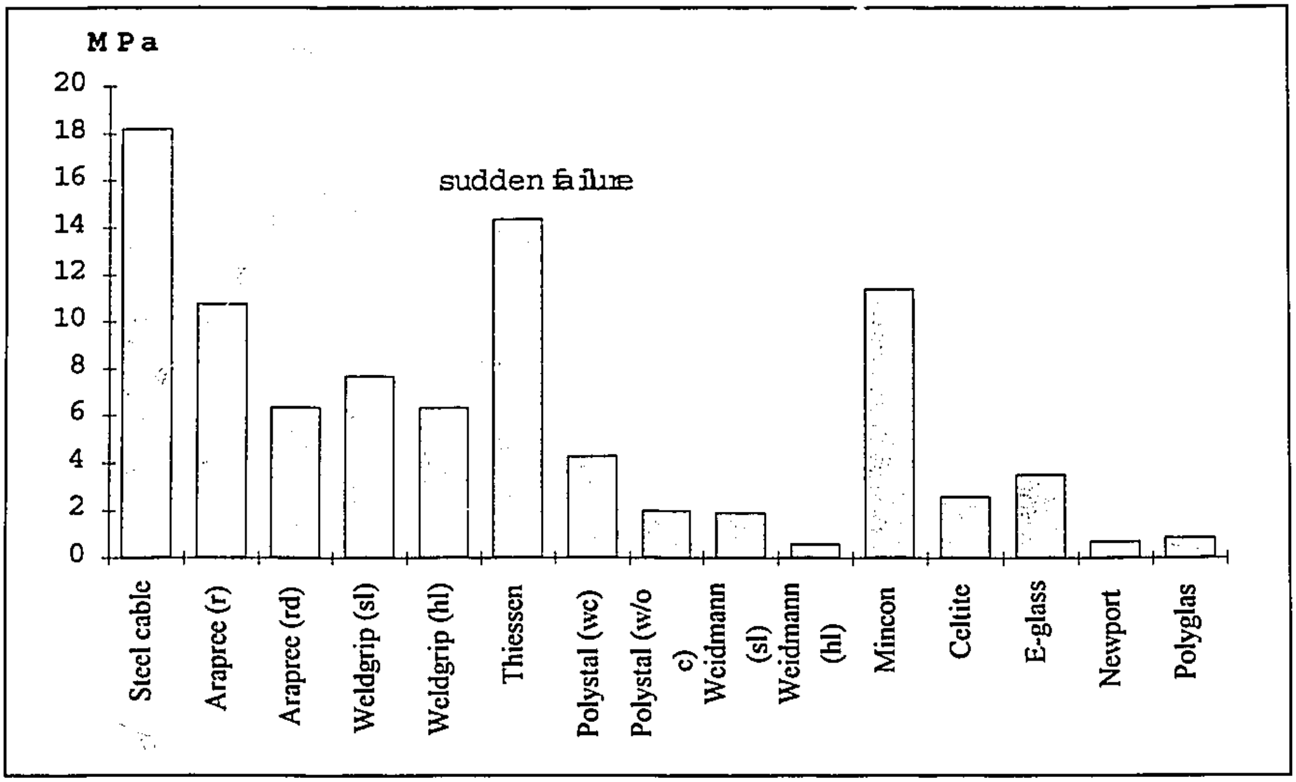
> *"For a given mine, it is possible that the use of more rigid support obtained by increasing the diameter of the mechanically anchored bolts or resin-grouted bolts would make it possible to reduce the incidence of broken bolts. However, the experience acquired in locations where rock bursts are likely to occur shows that the mine will eventually have to opt for a more yielding type of support, more compatible with the amount of energy released by the ground that must be absorbed by the support"*

For a given mine, it is obvious that in future, in rock mass classification systems, in addition to existing parameters, the ductility of both bolt material and grout-bolt interface will be mentioned. In some cases, a more brittle type of support system may be required and in other situations a more ductile reinforcement may be suitable. Composite tendon testing shows that, at the same time, the critical bond length of less than 100 mm in one type and of a few meters in another type can be possible. Moreover, the elongation capacity of composite bolts can be varied at the manufacturing process. From the literature and laboratory data, it would seem to be possible that in future, a bolt compatible with the design requirements, derived from the assessment of the deformational behaviour of rock mass, can be achievable. Similarly, the property of the 'cuttability' of composite tendons should encourage further the development of continuous mining based on a continuous support system.

Table 7.1: Diameter and tendon-grout interface shear strength of various composites

Tendon type	Tendon dia (mm)	Tendon breaking load (kN)	τ_{max} (MPa)
Conventional steel cable	15.5	240	18.21
Arapree (rectangular)	20 x 5	133*	10.8
Arapree (round)	8.0	66*	6.35
Weldgrip (solid)	22	300*	7.67
Weldgrip	22	300*	6.33
Thiessen	22	120	14.34
Polystal (with cover)	8.5	65*	4.3
Polystal (without cover)	6.0	40*	2.0
Weidmann (solid)	22	200*	1.92
Weidmann (hollow)	20	160*	0.58
Celtite	26.5	nr	2.58
Mincon	22	nr	11.40
E-glass	3.3	nr	3.51
Newport	30	nr	0.692
Polyglas	25.4	nr	0.85

nr = not reported; * Data provided by the manufacturers



r = rectangular; rd = round; sl = solid; hl = hollow; wc = with cover; w/o c = without cover

Figure 7.20: Average maximum shear strength of tendon-grout interface of various types of composite materials

An important concern in every field where composite products are gaining acceptance is the lack of available design criteria and, even more importantly, of standard test methods and equipment. Mining R & D should also focus on initiatives to introduce, develop and produce design guidelines for composites. Such initiatives should move the composite manufacturers towards mining applications needs.

On the basis of these findings, Arapree and Weldgrip tendons are further studied. Laboratory investigation on their pull out behaviour is presented in detail in the following chapters.

Chapter 8

Laboratory Investigation on Arapree Composite Tendons

8.1 Introduction

ARAPREE is an ARAmid PREstressing Element made of Twaron continuous aramid fibers pre-impregnated in epoxy resins. Arapree tendons are manufactured in both round and rectangular strip shapes with sanded and helical surface profiles. Arapree tendons have been used as tension elements to prestress concrete structures. The modulus of elasticity of Arapree is 125 GPa (steel = 200 GPa) and the ultimate tensile strength based on net cross-section of fiber contents is approximately 2800 MPa (prestressing steel = 1700 MPa). Pull out tests have been conducted on fully grouted. Arapree tendons using the same test setup of steel cables. Both cement and polyester resins were used as grouting materials. Additionally, modified configurations of Arapree tendons such as twin parallel and twin zig-zag were also tested.

8.2 Experimental Program

Laboratory investigation was conducted on fully grouted Arapree cable in the conventional steel cable test setup and the grouting material was selected as conventional grout (cement type 10 + water), high strength cement grout, shotcrete grout. A water/cement ratio of 0.4 was kept constant for all types of grout. These conditions are more suitable to bolting in hard rock mining The embedment length was kept constant as 300 mm for the purpose of comparison with each other and with the work of other researches (for example, Goris, 1988). A second laboratory test program was conducted with polyester resin grout, used as grouting material, to simulate rock bolting conditions in soft rock mines such as potash mines.

8.3 Pull Out Tests with Cement Grout

Conventional cement grout is used successfully as a bonding agent with cable bolts and rockbolts. But, conventional cement grout takes at least 7 days to cure. The effectiveness of a support to control excessive deformations in the of rock mass can be substantially reduced due to this time delay. High strength cement (ordinary cement + water + silica fume + additives) was, therefore, included and tested in the experimental program. Details are given in Section 3.4, along with test results, which show that high strength grout gains approximately the same strength in two days as conventional grout in 28 days. The ductility, however, is still maintained; the results in the following sections show that a displacement of 100 mm was achieved. A shotcrete grout is a combination of coarse aggregate (passing sieve size 9.50 mm (US 3/8) and retaining sieve size 2.36 mm (US 8)) and cement. This increases the radial stiffness of grout, hence, the load transfer mechanism is believed to be improved.

Methodology

Conventional pull out testing technique was followed in the laboratory experimental program. Arapree tendons were grouted in schedule 40 steel pipe, which was used to model drill holes in the host rock mass. A steel pipe with inner diameter of 57 mm and embedment length of 305 mm was selected as host medium. Water cement ratio of 0.4 for cement grout was kept constant for all the samples. All types of grout were tested after 28 days of moist curing. Minimums of three samples were prepared for each category of Arapree tendon. One test result, which was close to an average of three was selected to document in the report. An average (out of three) was not calculated, since it conceals the real behaviour of pull out tests such as chemical adhesion and elastic response prior to peak-failure and post-peak "stick-slip" response. Special anchoring device (grips, chucks, etc.) to hold the Arapree tendons (see Section 4.9) was used in the experiments.

The tests were conducted under constant displacement and rate of the displacement increments was selected to be small enough (i.e. 1 mm/min) to reflect the non linearity of the response for bond stress between Arapree tendon and surrounding matrix. The number of increments was large enough to provide approximately 100 mm displacement for each test. Different configurations of Arapree cable such as single Arapree tendon, twin zig-zag Arapree tendon and twin parallel Arapree tendons, were investigated.

Comparison between Fiber and Composite Tendon Pull Out Behaviour

The pull out behaviour of flexible composite tendon is slightly different from the steel cable. This difference is mainly seen in the post-peak region of the pull out experiments. Composite tendons pull out in an "oscillating" behaviour. While, steel bolts have no such behaviour. An analogy is found in the pull out experiment of single fiber from the grouted medium. The following is the description of this analogy, extracted from the work of Hull (1981) and Broutman (1970).

> *"Experimentally, it is found that for fibres embedded in brittle polyester and epoxy matrices, the stress required to extract the fibre does not drop to zero after debonding has occurred because there are large frictional forces which resist the sliding of the fibre out of the resin sheath. This is best illustrated in an*

experiment as shown in Figure 8.1. A short length of the fibre is bonded to a disc of the resin and the load required to pull the fibre through the disc is measured. The load-displacement curve shows that there is a peak associated with debonding and that after debonding an approximately constant load is required. The frictional forces are usually attributed to residual stresses associated with resin shrinkage during curing and to differential thermal contraction. In addition the reduction in the stress in the fibre at debonding results in an increase in fibre diameter owing to the Poisson expansion and this leads to an increase in the pressure on the fibre surface. the relative contribution of the interface bonding and frictional forces to the work required to pull the fibre out of the matrix depends on a large number of material parameters. However, it is clear from Figure 8.1 that the frictional forces could make a major contribution if the distance over which pull out occurs is large."

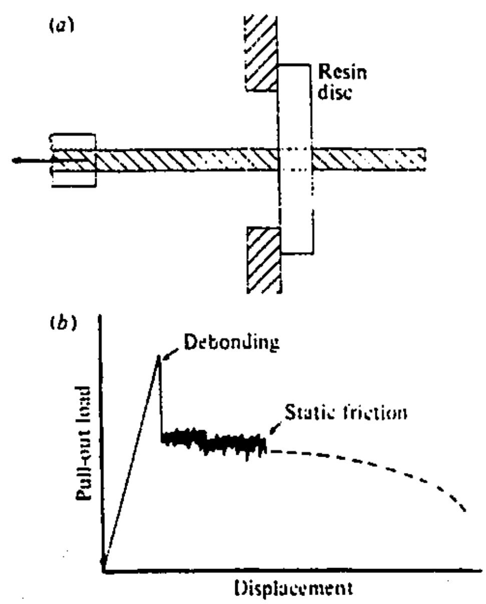


Figure 8.1: Pull out test (a) fibre embedded in resin disc, (b) typical load-displacement curve (Broutman, 1970)

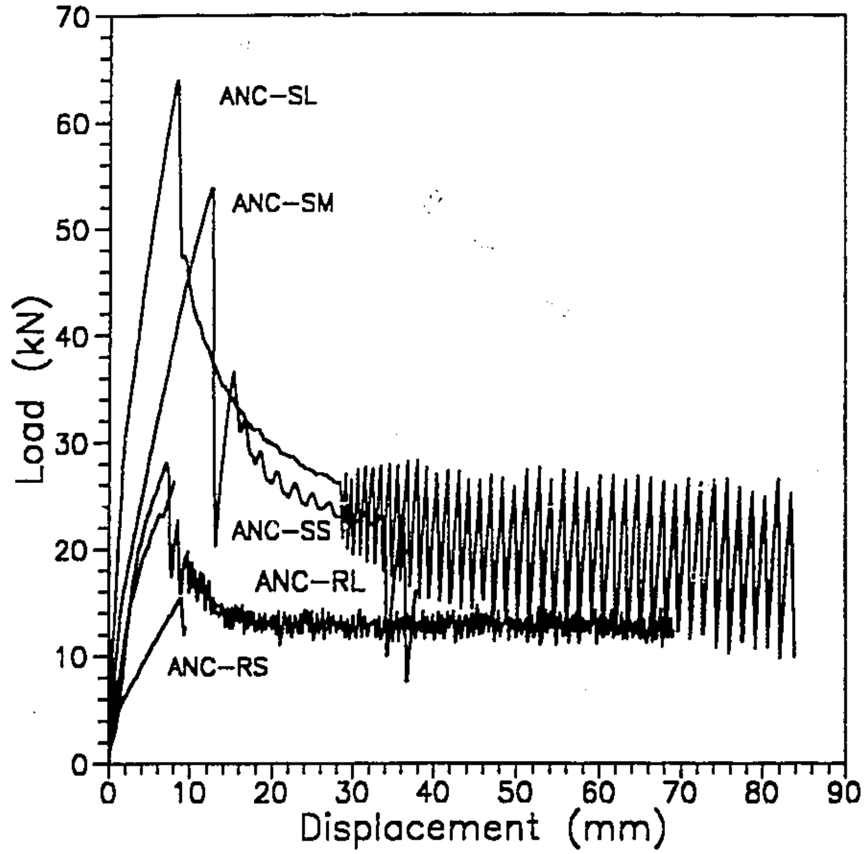


Figure 8.3: Pull-out behaviour of various types of Arapree tendons in conventional grout

On the basis of these results, only rectangular f400000 Arapree was selected for future testing and rectangular f400000 Arapree tendon will be referred to as Arapree tendon for the reminder of the chapter. Tests of single Arapree were conducted in conventional grout, high strength cement grout and in shotcrete grout. In the same test setup, a conventional steel cable was also tested to compare its results with the Arapree tendon. Figure 8.4 shows pull out response of tendons in conventional grout and Figure 8.5 in high strength grout.

Table 8.3: Test parameters and pull out test results of Arapree and steel cable

Sample Type	Cable Type	Grout Type	$P_{ultimate}$ (kN)	$P_{residual}$ (kN)
NCS-C	Steel	Shotcrete	72 ± 5.2	82 ± 5.8
NCS-0	Steel	Conventional	50 ± 4.3	60 ± 5.3
NCA-C	Arapree	Shotcrete	80 ± 4.4	30 ± 3.0
NCA-0	Arapree	Conventional	64 ± 3.8	25 ± 3.6
HSS-0	Steel	High Strength	68 ± 6.6	120 ± 12.4
HSS-C	Steel	Shotcrete-H	60 ± 4.4	66 ± 8.2
HSA-0	Arapree	High Strength	82 ± 4.4	32 ± 4.5
HSA-C	Arapree	Shotcrete-H	60 ± 5.8	20 ± 4.4

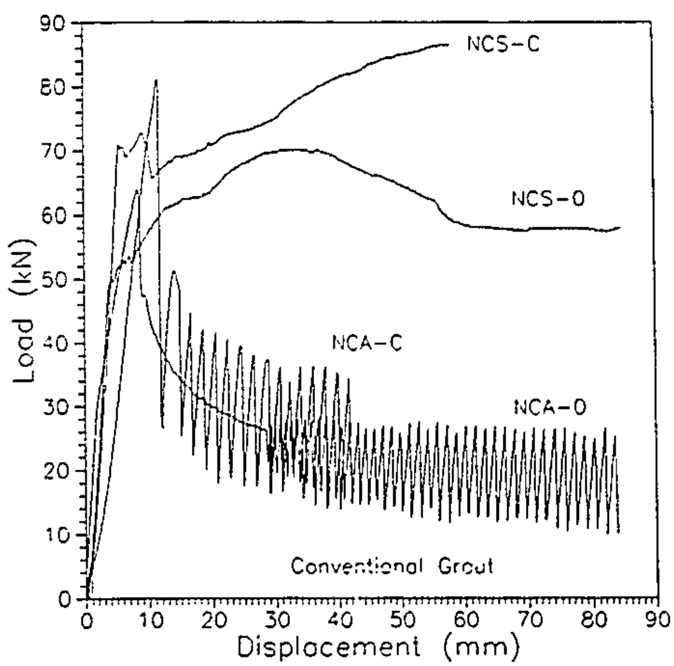


Figure 8.4: Pull out behaviour of Arapree and steel cable embedded in conventional and shotcrete cement grouts

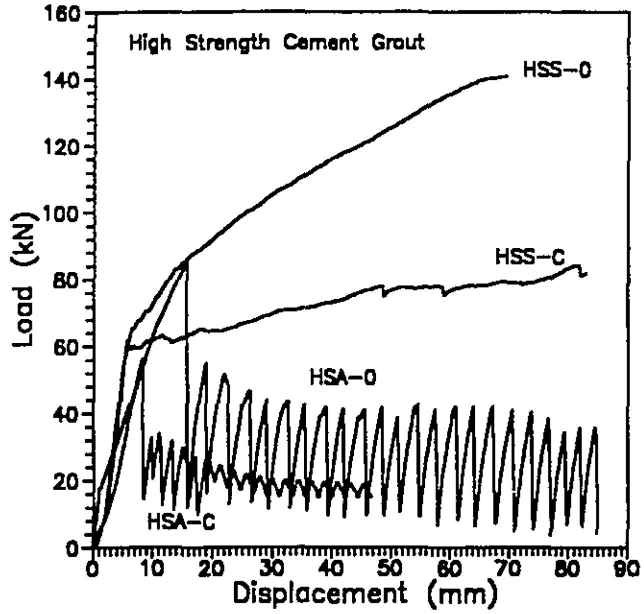


Figure 8.5: Pull out behavior of single Arapree and steel cable embedded in high strength and shotcrete cement grouts

The following conclusions can be drawn from these results:

1. Like steel cable, Arapree tendons first deform along with the grout i.e. perfect bond deformation.
2. After initial debonding, the residual strength of the tendon largely depends on the frictional resistance between the tendon and grout.
3. Performance of tendon support system increases with high strength cement and shotcrete grout.
4. From the visual inspection of pulled out sections of the Arapree tendon, no failure of any fiber or outer skin was observed.
5. The post-peak resistance of Arapree is less than a steel cable. This capacity stays within a range of 10 to 40 kN. The post-peak "stick-slip" behaviour can be explained as 'static friction' between tendon and grout, see Section 8.3.

8.3.2 Twin zig-zag Arapree Tendon

From the literature (Goris, 1990), it is found that the configuration like the birdcage of a conventional steel cable improves the resistance capacity of cable when pulled out from the grout. The reason is that in addition to mechanical and frictional bonding between cable and grout, the compressive strength of grout (the grout which gets confined in the cage of steel cable) also directly influences the pull out resistance. Two strips of Arapree tendons were constructed in the zig-zag formation in the laboratory. See Figure 8.2. Samples were tested in both conventional grout and high strength cement grout. The node spacing was kept constant at 15 cm (half the embedment length) for all the experiments. The effect of node at different locations is also considered. The parameters for the experiments are given in Table 8.4. The graphical representation of the pull out behaviour of twin zig-zag Arapree tendons is shown in Figure 8.6.

Table 8.4: Test parameters of Arapree twin zig-zag tendon

Sample Type	Grout Type	Top Node	$P_{ultimate}$ (kN)	$P_{residual}$ (kN)
TZAR-CA	Conventional	Anti-node	78 ± 4.9	30 ± 2.3
TZAR-CN	Conventional	Node	46 ± 4.4	18 ± 2.2
TZAR-HA	High Strength	Anti-node	60 ± 5.9	70 ± 7.7
TZAR-HN	High Strength	Node	68 ± 3.8	30 ± 5.8

TZ = Twin zig-zag; AR = Arapree; C = conventional cement grout; H = high strength cement; A = anti-node; N = node

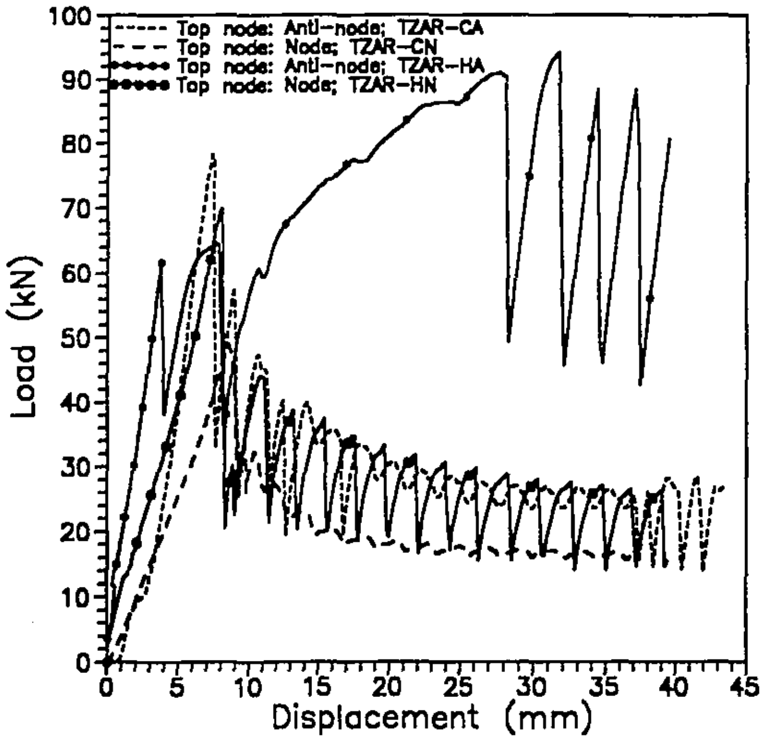


Figure 8.6: Pull out behaviour of twin zig-zag Arapree tendons

The following conclusions can be drawn:

1. The analogy of steel birdcage cable is not identical to the Arapree twin zig-zag configuration. The reason is that the Arapree strip slips through the node and the grouted confined area after mechanical debonding,. It does not compress concrete in between strip openings and as a result the performance does not improve. The other possible reason for this type of behaviour is that the stiffness per unit length of Arapree is relatively higher than the grout, therefore, the off-axis loading on Arapree break the grout wedges before complete debonding. This weakens the system and Arapree twin zig-zag tendon pulls out prematurely. This failure, however, is due to a loose knot (anti-node), which was manufactured manually in the laboratory. The manufacturers of Arapree tendon have been informed about the requirements for

knot. A knot that would be produced during the manufacturing process by criss-crossing the fibers will not let the Arapree slip at the anti-node and hence, a steel birdcage cable like response can be expected.

2. The visual inspection of pulled out birdcage Arapree tendons show that the Arapree tendons remain unaffected i.e. the applied stress was below its strength.
3. The position of the node does not make any significant effect on the pull out behaviour.
4. Few tests, however, showed unusual behavior; as shown in Figure 4.20. The mechanical debonding started at about 60 kN and stayed between 50 to 90 kN.
5. It seems that there is no benefit of manufacturing Arapree in zig-zag configuration. If the knot is not stronger than the compressive strength of the grout, the pull out response will not improve.

8.3.3 Twin Parallel Arapree Tendon

A twin parallel Arapree configuration was developed with the same concept of confining the grout in between the Arapree tendons and the failure of the grout-tendon interface was expected to begin after exceeding the compressive strength of grout. An aluminum spacer (button) was inserted between strips to make a node. No special grip or anchoring device was available to hold this type of configuration, therefore, the ordinary anchoring device (chuck) was used to pull out the Arapree tendon. This chuck failed at about 120 kN of load. The grouted portion of the tendon and the grout, however, remained unaffected. The parameters of testing are given in Table 8.6 and the behaviour is shown graphically in Figure 8.7. The following conclusion can be drawn:

1. Since the loading was axial on the tendons, therefore, the obtained pull out behaviour of grouted tendons should have been identical to single Arapree tendon failure had there been a better anchoring device.

2. Straight twin parallel tendons increase the capacity of the support system.
3. The cost of overall support system increases with the use of such configuration.
4. High strength grout did not improve the capacity. More research is required in the field of high strength cement grout to find the reason of its premature failure.

Table 8.5: Test parameters of parallel Arapree tendon

Sample Type	Grout Type	Failure Mode	$P_{ultimate}$ (kN)	$P_{residual}$ (kN)
TPAR-C	Conventional	Failure of anchoring device	115 ± 5.1	NA
TPAR-H	High Strength	Failure of cable-grout interface	80 ± 5.0	17 ± 4.1

TP = Twin parallel; AR = Arapree; C = conventional cement grout; H = high strength cement grout

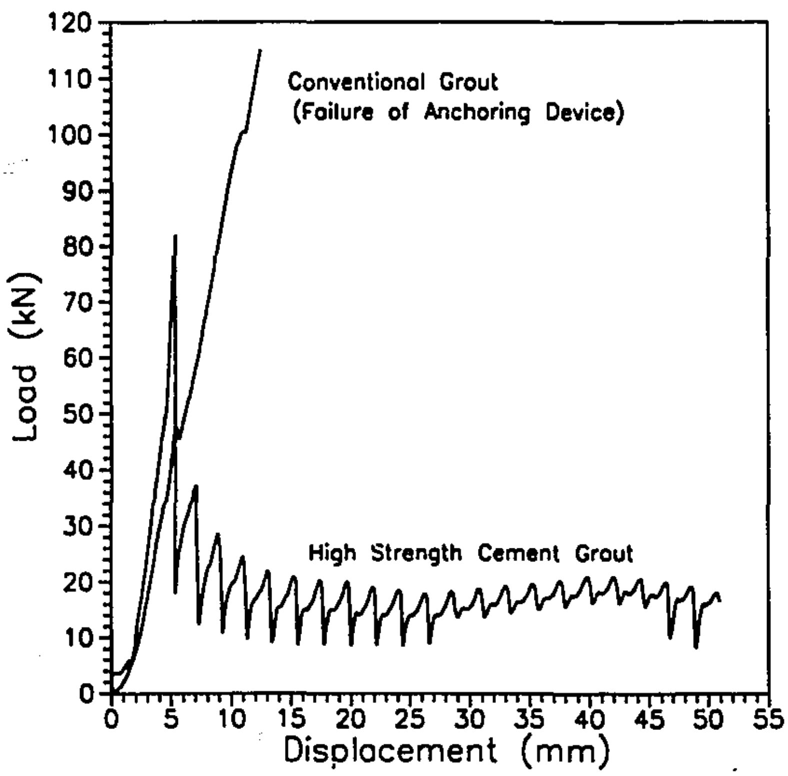


Figure 8.7: Pull out behaviour of twin parallel Arapree tendons

8.3.4 Shear Tests on Fully Grouted Arapree Tendon

Single Arapree cable was tested for its shear capacity by embedding it in cement grout. Schedule 40 steel pipes were used to simulate the drilled hole in the host medium. A 4 mm space was left between the two pipes to facilitate the adjustment of the sample in the shear box. One steel pipe was attached and fixed with the base of R.D.P while other was moved upward to induce a shear force in grouted Arapree cable (see Picture 8.2, for detail). The shear force versus shear displacement is illustrated in Figure 8.8. The following conclusion can be made:

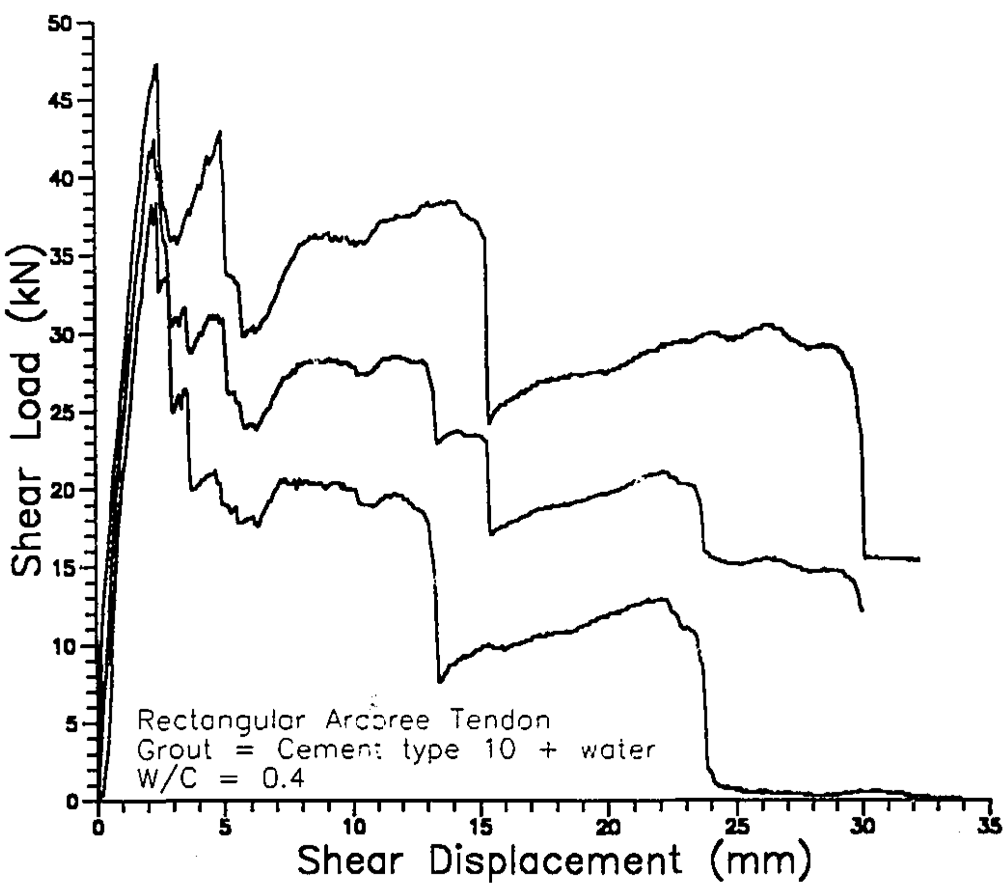


Figure 8.8: Shear test on cement grouted Arapree tendons

1. The average peak pull out capacity, as shown in the figure, is only 40 kN. The failure started simultaneously with the generation of cracks in the grout and shearing of the fibers.
2. Unlike shear tests with resin grout (see section 8.4.4), the load was initially distributed evenly on the entire cross-sectional area of the Arapree.
3. The flexibility of Arapree cable does not make the shearing process more ductile. In other words, more displacement of Arapree reinforced shearing zone should not be expected considering the flexible behaviour of Arapree. In fact, Arapree started to fail at about 5 mm of shearing displacement. The average residual shearing load carrying capacity is only 20 kN with 25 mm of shearing displacement.

8.4 Pull out Tests with Resin Grout

Cement based grouts can not be used in soft rock mines such as potash mines. Organic grouts such as Polyester or epoxy resin grouts are used, in addition to mechanical and frictional bolts, to embed the tendons to support or to reinforce ground in salt mines. Therefore, resin fully grouted Arapree tendons were investigated in the same test setup as was used in the cement grout. There are, however, fundamental differences in the behaviour of resin and cement grout during the pull out process i.e. resistance to shear stresses at the bolt-grout interface.

Resin grout was manually mixed in the laboratory. Then, thoroughly mixed resin was poured into the steel pipe. The tendon was inserted in the steel pipe before mixing the resin. The initial setting time of resin was 5 to 7 minutes. Therefore, except for small length samples, resin was mixed and prepared for one sample only, unlike cement, where cement was prepared for at least three samples. However, a minimum of three samples were prepared for each category.

Methodology

Different embedment lengths of the Arapree tendon were prepared to investigate the effect of embedment length on pull out capacity. R.D.P Servo Control Stiff Testing Machine has a capacity to accommodate maximum of 1000 mm embedment length samples with only 145 mm of anchoring length. Different configuration of Arapree tendons required two-pipe pull out grouting technique, which was seldom possible with the RDP Testing facility. Therefore, a Hydraulic Jack (also known as Hydraulic Cylinder) was used to test the rest of experiments. Picture 8.3 shows the arrangements of pull out test with the Hydraulic Jack. Displacement was initially measured by both Digital Vernier Calipers and a Potentiometer. Later on, only a Digital Vernier Caliper was used for this purpose and the obtained measurements were found accurate upto ± 0.01 mm.

In order to calibrate and compare the Hydraulic Jack pull out behaviour with the RDP testing equipment, fully resin grouted conventional steel cable samples were prepared and tested in both systems in identical conditions. Figure 8.9 and 8.10 shows pull out response with the RDP Servo Control Testing machine and the Hydraulic Jack, respectively. From these figures, it can be concluded that there is no significant difference between the data obtained and these techniques can be generalized for each other.

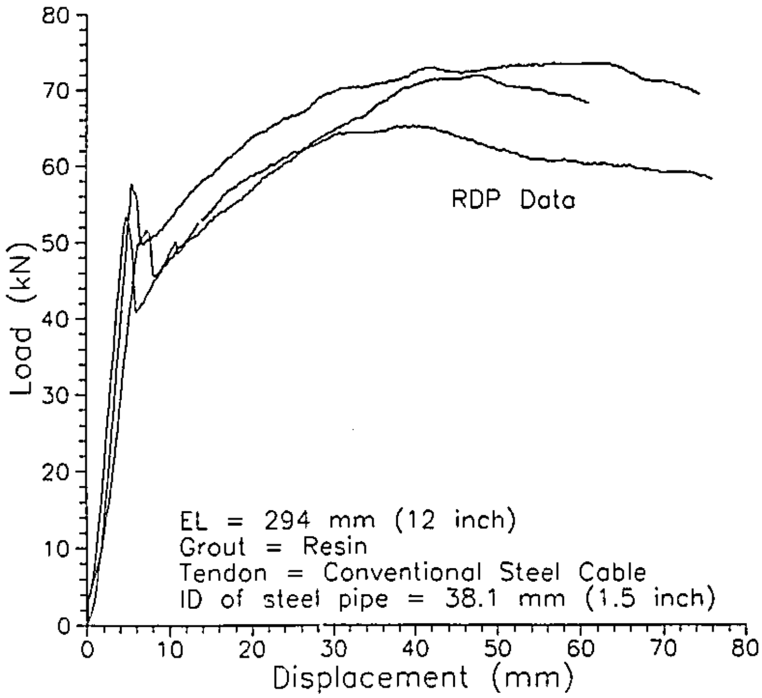


Figure 8.9: Pull out response with RDP Servo Control Stiff Testing Machine

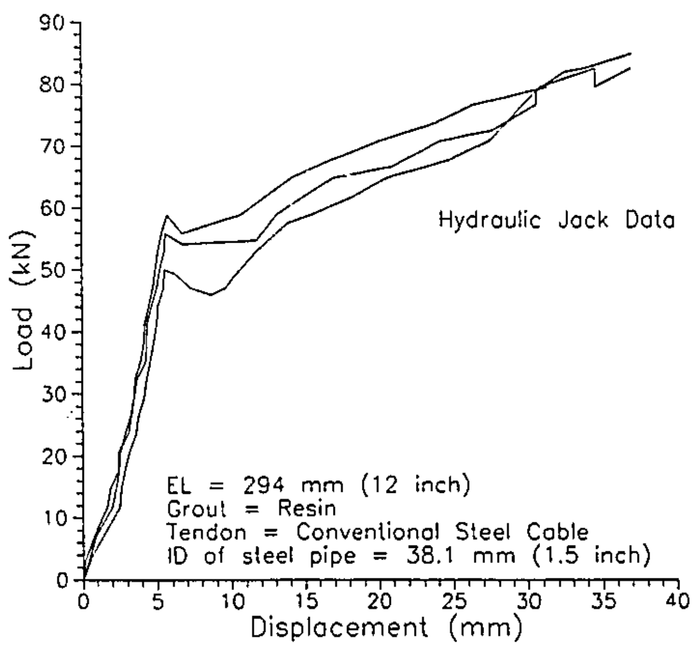


Figure 8.10: Pull out response with Hydraulic Jack

Picture 8.3: Hydraulic Jack test set-up

8.4.1 Observations of Failure Mechanism

Resin fully grouted Arapree tendons behave differently than cement fully grouted Arapree tendons. The following are the probable reasons for such unanticipated failure mechanism and premature pull out behaviour of Arapree tendons:

1. Arapree is rectangular tendon in cross section. When embedded in circular steel pipe, the amount of grout between Arapree and steel pipe is, obviously, not uniform, as shown in Figure 8.11. This uneven configuration, did not influence the results of pull

out tests with cement grout, however, with resin grout, a very significant effect is observed. As mentioned in Section 3.6.1, amount of resin should be between 1.2 to 1.5 of drill hole / tendon dia. If it is less, then resin will not have enough space to make a bond between the cable and the drillhole. On the other hand, if the resin quantity is more than this ratio, then resin will shrink and will induce micro cracks. As obvious from Figure 8.11, the amount of resin filled in between the cable and the steel pipe (width direction) was more than the specified quantity. This led to the shrinkage of the resin in that direction and caused the premature pull out of the Arapree tendon.

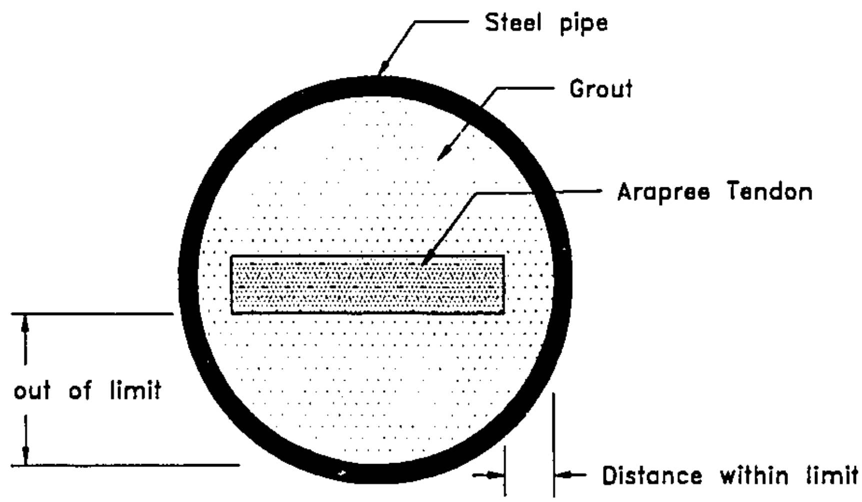


Figure 8.11: Cross-sectional view of embedded Arapree in steel pipe

2. From the observations, it is noticed that resin does not behave as brittle during the pull out test as it behaves under uniaxial compressive load. During the pull out process, resin deforms radially or bulges out. This caused the failure of the bond between bolt and grout prematurely. Whereas, resin cylinder samples failed just like cement samples under compressive load and no bulging was observed. This

discrepancy is most likely associated with the encasing of resin samples during the curing process. Resin might have not cured completely when poured into steel pipes along with tendons.

3. The construction and material of steel cable and Arapree are fundamentally different from each other, as shown in Figure 8.12. Both these tendons contract diametrically during pull out tests. However, the rate and amount of contraction are different, and so does the pull out behaviour. Same radial contraction problem was faced by Stillborg (1984), during his pull out tests of steel wire ropes. (The capacity of contraction in steel wire rope is more than steel cable, because of its construction and diameter of steel wires). It was observed during the pull out tests that Arapree tendons tend to contract radially under axial load. This factor also added in their premature failure. The conceptual description of Arapree tendon diametric contraction during the pull out test is shown in Figure 8.13.

8.4.2 Single Arapree Tendon

Various embedment lengths such as 100 mm (4 inch), 200 mm (8 inch), 305 mm (12 inch) and 406 mm (16 inch) were tested to investigate the effect of embedment length on their pull out behaviour. Schedule 40 steel pipe with inner diameter of 38.1 mm (1.5 inch) was used as host medium. Arapree cable chucks were used to grip the Arapree cables and they were found successful in gripping them. The following are the results and observations of the tests:

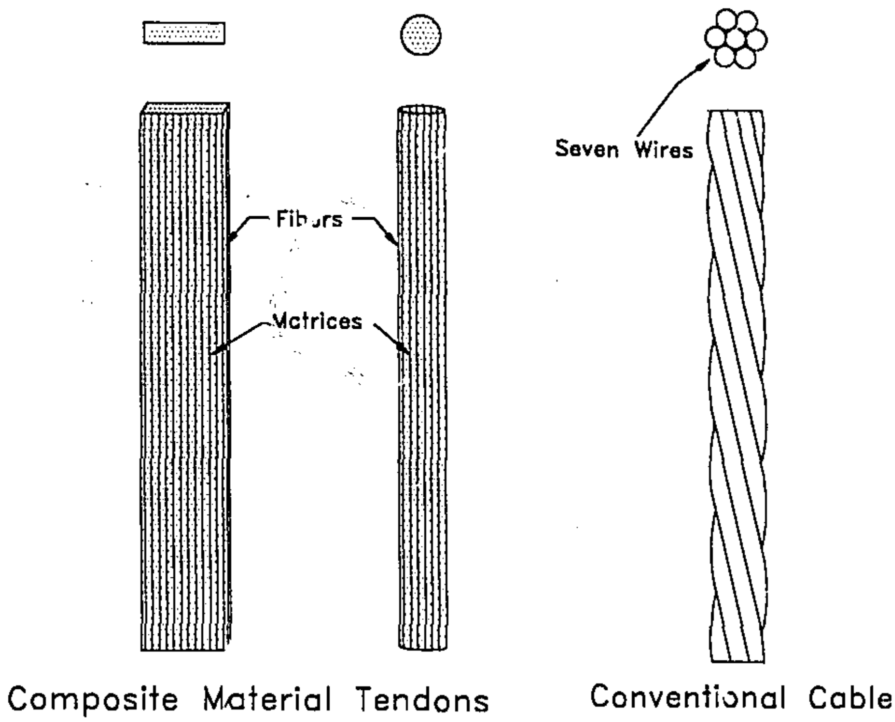


Figure 8.12: Structural difference between composite material tendons and conventional steel cable

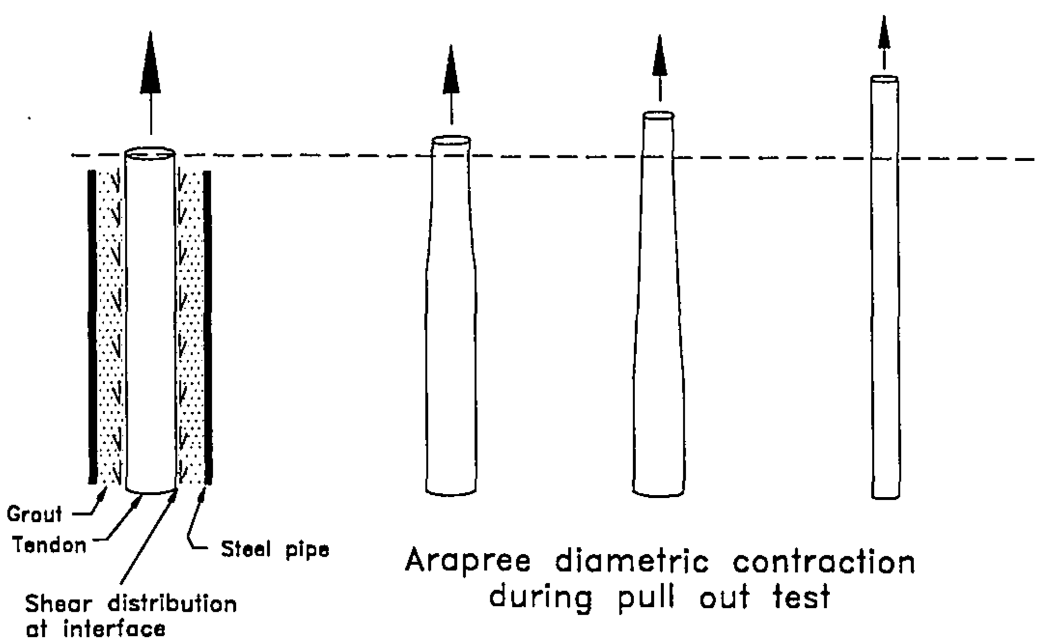


Figure 8.13: Conceptual diametric contraction of Arapree tendon during pull out test

1. All the above mentioned fully grouted embedment lengths i.e. 100 mm (4 inch), 200 mm (8 inch), 305 mm (12 inch) and 406 mm (16 inch) failed or pulled out prematurely. The load carrying capacity was less than 25 kN in all the embedment lengths. This capacity does not increase linearly with the grouted length bolt. See Figure 8.14.
2. A longer embedded length of 785 mm (31 inches) was then tested, but it also failed prematurely.
3. In order to confirm the results, an Arapree cable was embedded in cement grout by keeping the same resin grout test setup. The embedment length was 785 mm (31 inch). An expected result was obtained as shown in Figure 8.15. The peak pull out load was 108 kN at 30 mm displacement. The residual load carrying capacity remained between 20 - 40 kN.

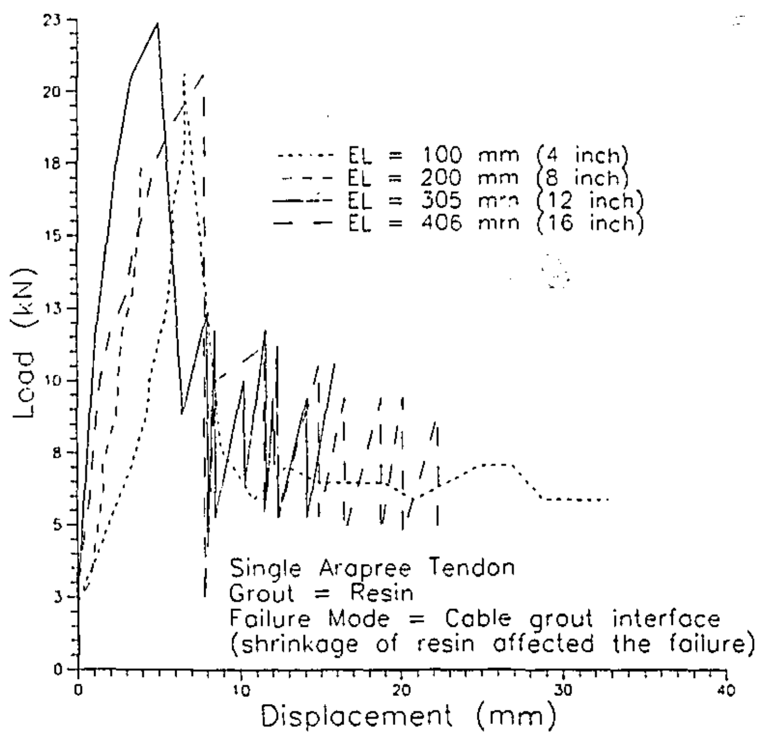


Figure 8.14: Pull out response of various embedment lengths of resin grouted single Arapree cables

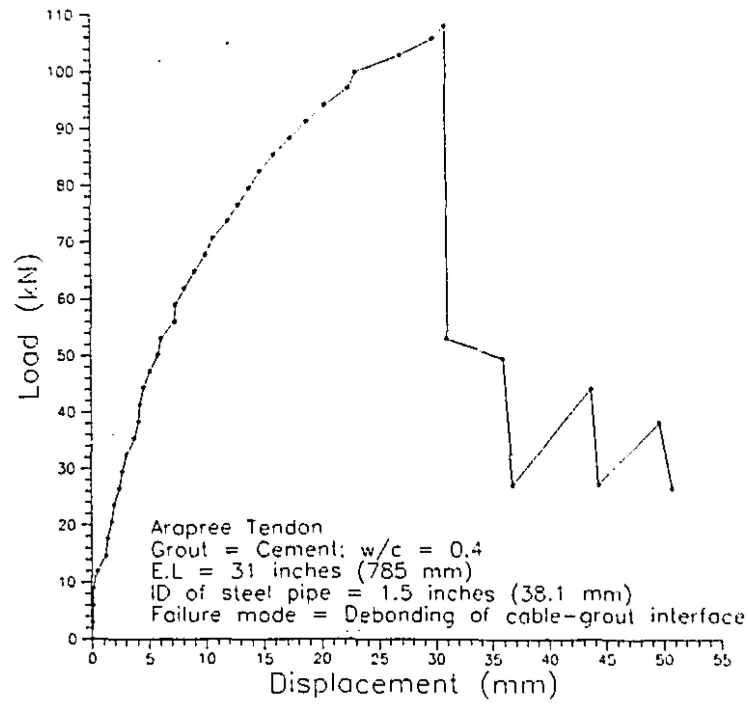


Figure 8.15: Single Arapree embedded in 785 mm (31 in) long cement grouted steel pipe

8.4.3 Twin Parallel Arapree Tendon

Same embedment lengths, as tested with single Arapree tendon, were taken as one of the variables in these tests. Two parallel Arapree tendons with spacers at 100 mm (4 inch) distance apart, were constructed manually in the laboratory. The chucks or grips, used for single Arapree tendons, proved unsuccessful for twin parallel Arapree tendon configurations, therefore, tendons were grouted from both sides in two steel pipes. Enough spacing was left in between testing and anchoring steel pipes to adjust in the hydraulic jack. The intended testing embedded length was always taken as half the gripping length. In this configuration, it was made sure that only the testing length should slip or fail rather than the gripping length. The following are the results and observations of pull out experiments:

1. Almost identical pull out results were obtained as with single Arapree. Here, tendons also pulled out prematurely. The maximum pull out load was obtained as 80 kN with 406 mm (16 inch) of embedment length. See Figures 8.16 and 8.17.
2. Reasons for failure were the same, as pointed out earlier, except that, because of two tendons in the steel pipe, resin had less space to shrink as compared to single Arapree in pipe. Therefore, the pull out loads were slightly higher than the single tendon.
3. Failure was initiated due to breaking of the inserted spacers. Had there been a better spacer, there would have been different results. However, more reliable and desirable twin tendon configurations could only be achieved if they were prepared during the manufacturing process of Arapree tendon.

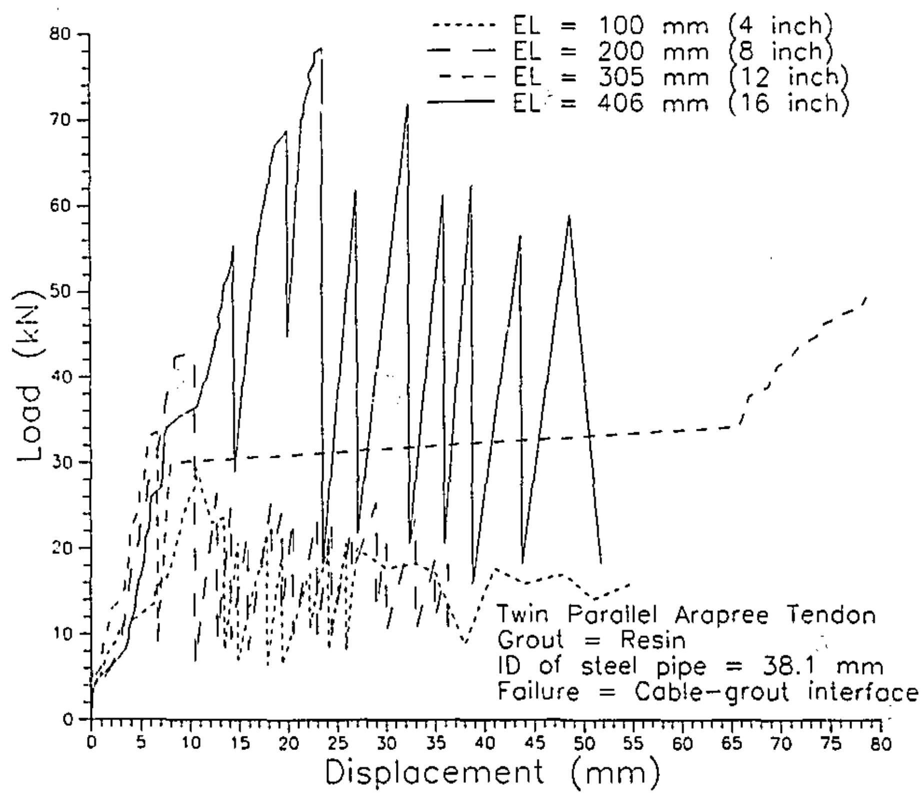


Figure 8.16: Pull out behaviour of fully resin grouted twin parallel Arapree tendons

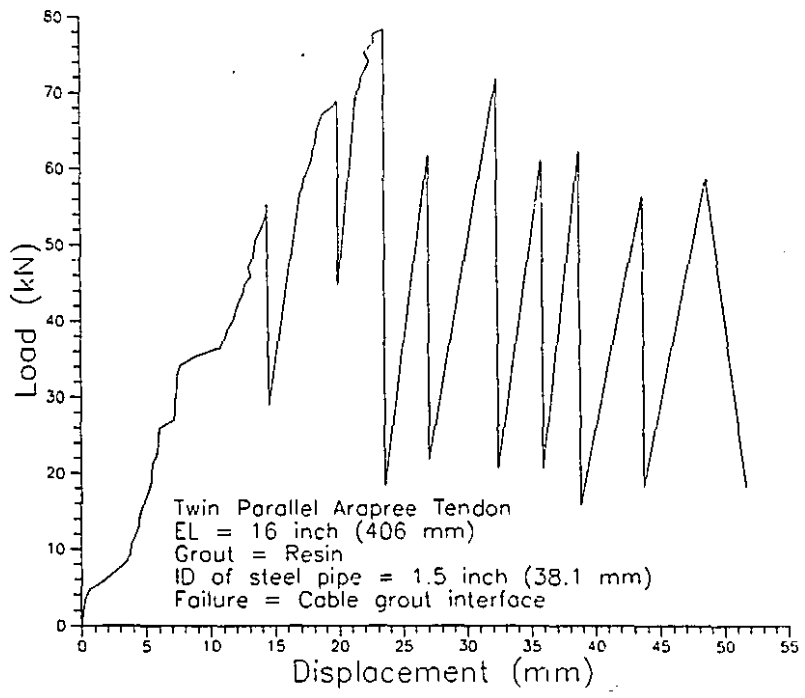


Figure 8.17: Pull out behaviour of resin fully grouted twin parallel Arapree tendon with an embedded length of 406 mm (16 in)

8.4.4 Twin zig-zag Arapree Tendon

Twin zig-zag Arapree tendons were constructed manually in the laboratory. Steel clamps were used to make nodes while aluminum spacers were inserted to produce anti-node configuration. The rest of the experimental procedure was identical with the twin parallel tendon tests. The following are the results and observations:

1. A pull out load close to 170 kN with 30 mm of displacement was obtained (see Figures 8.18). These results were expected due to this configuration of tendon.
2. The failure by slipping of the tendon was initiated at the node after tearing the steel clamps. Stronger steel clamps could have sheared the Arapree tendon, therefore, these were not used. A much better knot could have been obtained during the

manufacturing process of Arapree, which must be developed as a knot of criss-cross continuous fibers. However, this is the job of manufacturer of Arapree tendons.

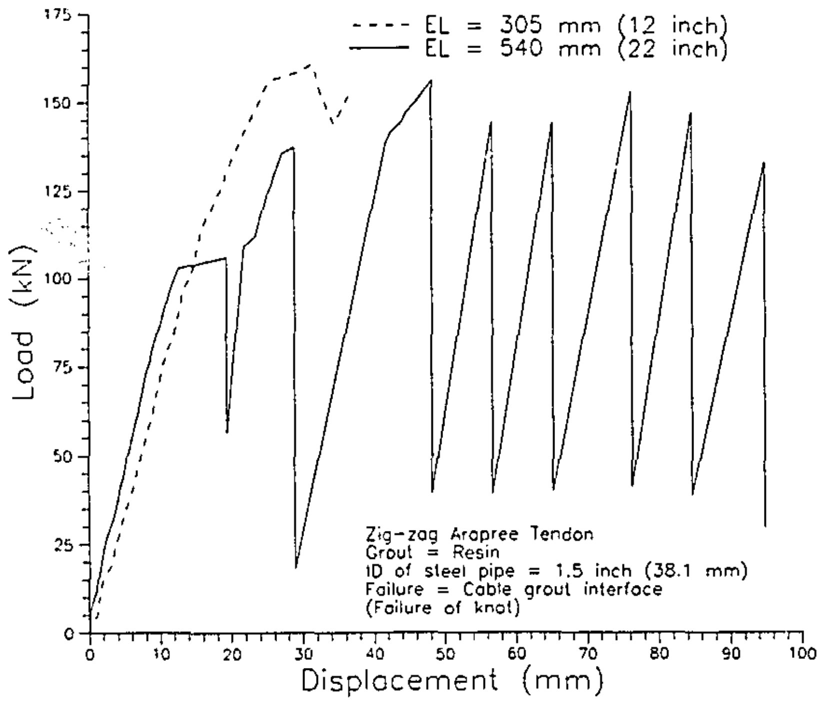


Figure 8.18: Pull out behaviour of twin zig-zag Arapree tendons with an embedded length of 305 mm (12 in) and 540 mm (22 inch)

8.4.5 Shear Tests on Fully Grouted Arapree Tendons

Arapree tendons, embedded in resin grout, were tested to evaluate its shear resisting capacity. The following configurations were used in the experiments:

1. Single tendon with the width parallel to the applied shearing load
2. Single tendon with the thickness parallel to the applied shearing load
3. Twin parallel tendon with the width parallel to the applied shearing load

4. Twin parallel tendon with the thickness parallel to the applied shearing load

The test results are shown graphically in Figure 8.19. From the results, it can be seen that shearing capacity of Arapree tendon is relatively very low, close to 30 kN. Results are identical in all four above mentioned configurations. Shearing capacity of Arapree did not improve with the modification of tendon configuration. This is due to the fact that shearing started due to breaking of individual fibers and load is not equally distributed over the entire area of the Arapree tendon during the shearing process.

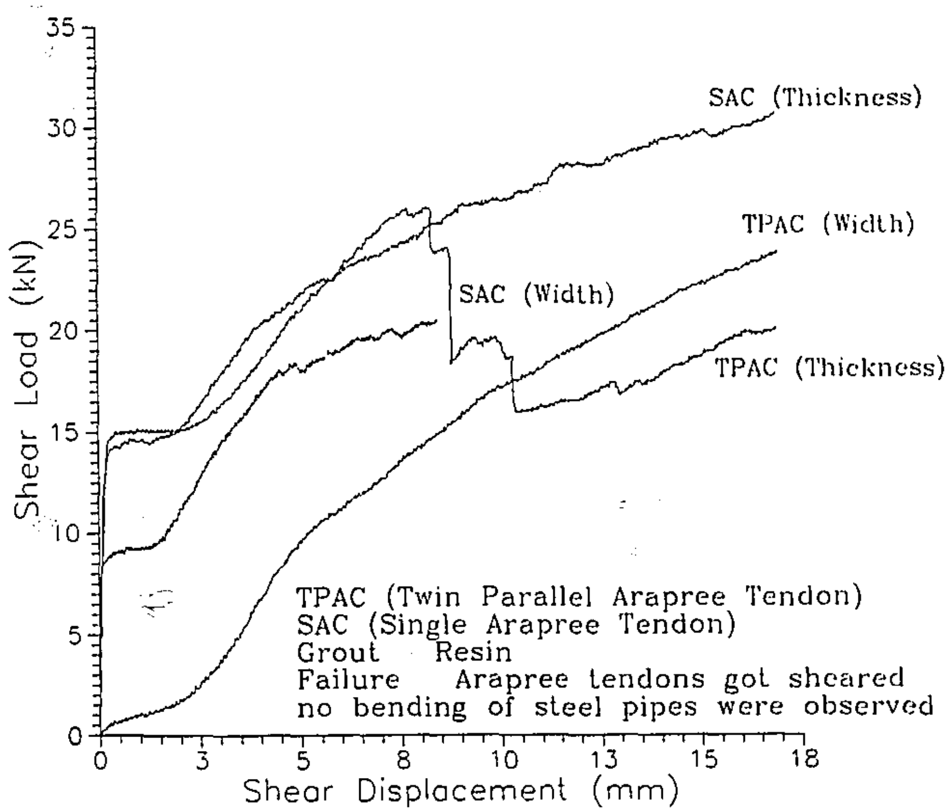


Figure 8.19: Shear test on resin grouted Arapree tendons

8.5 Discussion

Cement Grout

- The laboratory investigation show that Arapree tendons have potential to be employed in mining applications and that such composite materials are expected to be the future ground supports.
- In certain applications where the use of steel tendons has had drawbacks, the Arapree tendons are likely to offer a viable alternative.
- Pull out experimental results show that Arapree, embedded in cement grout, as a rockbolt has sufficient support capacity to be employed for reinforcing loose rocks near the excavated openings.
- The post-elastic limit of Arapree tendons is less than steel cable but is more than that of steel rockbolts. The residual capacity of cement fully grouted Arapree tendons is around 50 kN. This means that the rock mass supported by Arapree tendons should give warning prior to failure.
- In mining applications, cable bolts and rockbolts are designed considering their elastic limit under tensile and shear loading. Arapree as a cable bolt and/or rockbolt behaves like steel cable bolt in the elastic limit. Therefore, Arapree tendons can be designed to support the rock mass in place of steel cables.
- The capacity of fully grouted Arapree tendons can be improved by combining two Arapree tendons.
- Circular Arapree tendons are also commercially available. Some of them were tested here with cement grout. After manufacturing of the tendon, a roughness on their surface was created by gluing sand particles. These tendons pull out prematurely due to brushing off of this sand. More durable rough surface needs to be designed by the

manufactures. This would encourage experiments with circular Arapree tendon, which is more compatible than strip shape Arapree with existing mining bolting equipment.

Resin Grout

- Single Arapree tendon embedded in resin grout pulls out prematurely and its peak load carrying capacity appears to be very low. It is not recommended to use Arapree as reinforcing element with resin grout.
- A twin parallel Arapree bolt configuration pulls out from the knot, which was made in the laboratory. At the same time, shrinkage of grout and diametric contraction of Arapree becomes a reason for the premature pull out of the twin parallel tendons. This configuration is also not found very suitable for reinforcing purposes.
- Twin zig zag configuration of Arapree tendons sustained expected load. This was due to the improved interlocking between tendon and grout. Even then, failure initiated due to the failure of manually manufactured knot. Manufacturers of Arapree tendon have to come with strong knot. However, this series of tests proved that Arapree has capacity to be used as reinforcing element with resin grout, but it needs more research and development.
- Shear load carrying capacity of Arapree tendon is relatively very low. This is because of flexibility of tendon that led to the shearing of fibers gradually rather then distributing the shearing load on the entire cross-sectional area of the tendon. This is not surprising since fibers are known to resist mostly in tension.

General Discussion

- The mechanical properties of Arapree can be altered according to anticipated mechanical behaviour of rocks and, hence, a compatible support system can be "manufactured".

- The Arapree tendon is cuttable. This property encourages and contributes to the use of continuous mining method in both hard and soft rock environment.
- Arapree is a non-corrosive material, thus, it can be used in highly humid and hostile environments
- Arapree is anti-static, so it can be used near high tension power cables.

Chapter 9

Laboratory Investigation on Weldgrip Composite Tendons

9.1 Introduction

Weldgrip is made up of fiberglass and thermoset polyester resin. Pultrusion is the manufacturing process used to produce it. Weldgrip is a rigid circular tendon developed and produced exclusively for rockbolt support applications in underground mining. Hollow and solid types of Weldgrip rockbolts are commercially available. These bolts have a rough surface profile with two different types of surface patterns. This roughness is created by inducing impressions on partial curing of the bolts, immediately after passing through the pultrusion process.

The ultimate breaking load of Weldgrip rockbolt is more than 300 kN (breaking strength ≈ 960 MPa). Its nominal diameter is 22 mm. It comes in different lengths depending on the requirement. Its modulus of elasticity is not known, however from the other types of fiber glass tendons, it should be between 120 to 140 GPa. It is not an isotropic element and both strength and stiffness are different in both longitudinal and transverse directions. In the longitudinal direction, strength and stiffness depend on fibers and in the transverse direction, these properties depend on mechanical properties of the resin. Tensile load bearing is the basic applications of these tendons, however, these can also be used in compression load carrying applications. The former quality depends on fibers while latter on resin.

Weldgrip bolts are usually 8 feet (2.35 m) long. One end of the bolt, around 10 inches (245 mm) long, is provided with a threaded section to hold the nut and plate. The other end of the bolt is cut at a 45 degree angle to form a sharp edge. This end is used to tear and mix resin cartridges inside the drilled hole.

In order to evaluate the performance of Weldgrip tendons as rockbolts in underground mining applications, pull out tests were conducted on different embedment lengths with both cement and resin grouts. Two gripping methods were used to hold the Weldgrip bolts during the pull out process. First, only the threaded end with nut was used to hold the bolt and second, two-steel pipe grouting technique was used. The threaded portion, most often, failed between 60 - 100 kN of pull out load. In order to avoid this failure which was caused due to delamination of threaded section, extra nuts and clamps were used. These arrangements were found successful upto a pull out load of 150 kN. The two-steel pipe technique was also found successful. An embedment length of around 11 inches (270 mm) was kept constant for all the tests conducted with cement grout. The embedment length was intended to be 12 inches (294 mm) long, but, extra rubber stoppers were used to prevent leakage and bleeding of cement, which ultimately reduced the embedment length.

9.2 Pull out Tests with Cement Grout

The applications of cement grouted Weldgrip rockbolts are possible in all non-salt mines. Cement itself is smooth powder, therefore, it does not offer enough resistance to the slippage of the bolts, during the pull out process. Therefore, in order to resolve these problems, a combination of different types of cement based grouts were tried. A detail about these cements are given in chapter 3. These combinations are as follows:

1. Conventional cement with 0.4 w/c ratio.
2. Shotcrete grout with conventional cement.
3. High strength cement grout.
4. Shotcrete grout with high strength cement.

9.2.1 Pull Out Tests with Conventional Cement

Pull out results with conventional or cement type 10 are shown in Figures 9.1. The following are the results and observations of the pull out experiments:

1. All those samples whose gripping end was secured by additional clamps, failed by the debonding of he bolt-grout interface at around 118 kN of uniaxial load. This is only 40 percent of the breaking load of the tendon.
2. All those tendons, where threaded portion was used only as gripping device, were failed by delamination of outer glued threaded surface (gripping section) of the tendon. In this particular category, threaded end failed at around 80 kN.
3. In the elastic region i.e. prior to peak load, the pull out response is linear and identical to pull out response of conventional cement grouted steel bolts. In the post-elastic region, Weldgrip bolts have a residual load carrying capacity of around 20 kN. Which is generally not found in conventional steel bolts. This characteristic is, of course, associated with the surface pattern of the bolt.
4. By increasing the size of surface engraved knobs, the pre-elastic pull out response of the bolts can be increased. However, this will decrease the post-elastic load carrying capacity. On the other hand, a smooth surface pattern will have a vice versa effect.

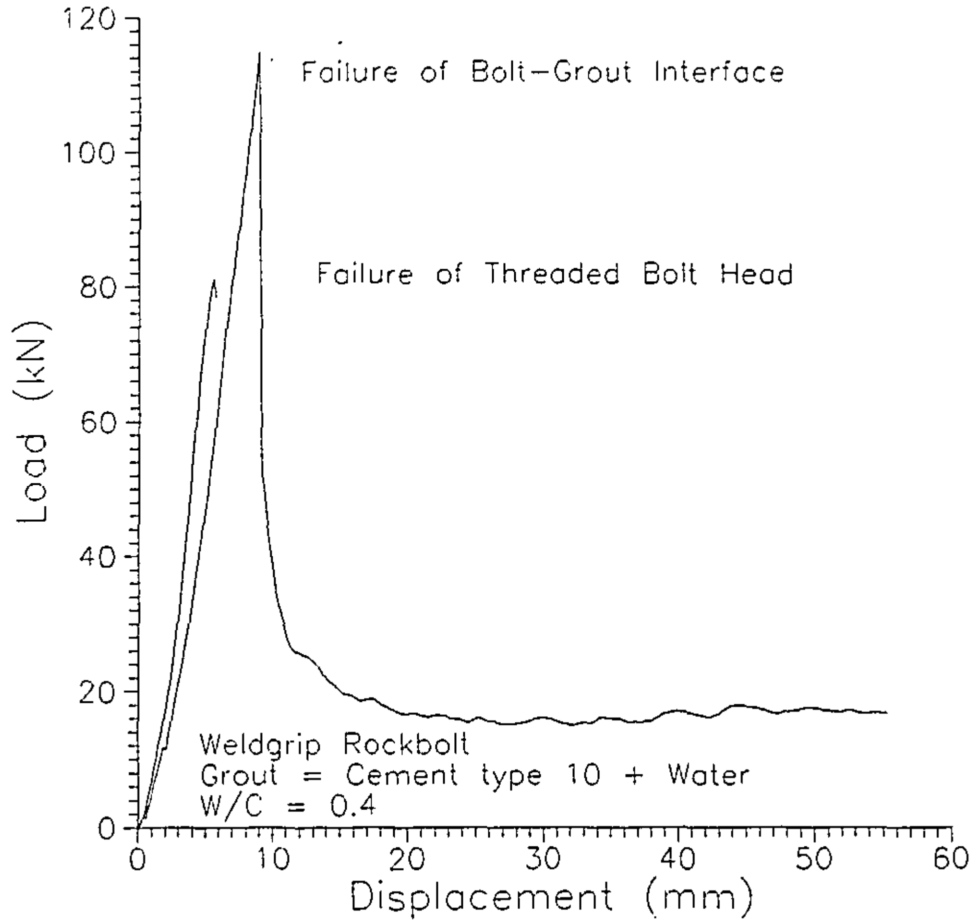


Figure 9.1: Pull out behaviour of Weldgrip bolts grouted with conventional cement

9.2.2 Pull Out Tests with High Strength Cement

Figure 9.2 shows graphically the pull out response of Weldgrip bolts fully embedded in high strength cement grout. The following are the results and observations of these experiments:

1. The pull out tests conducted with secured gripping system failed due to bolt-grout interface slippage (debonding). The maximum load carrying capacity was around 110 kN, which is approximately 37 percent of the bolt breaking load.
2.. Contrary to pull out tests with conventional cement, the threaded portion did not debond rather failure was occurred by bolt-grout interface debonding, as seen from the figure.
3. The overall response of pull out tests with high strength cement were just like conventional cement grout. i.e. pre- and post- elastic failure responses.

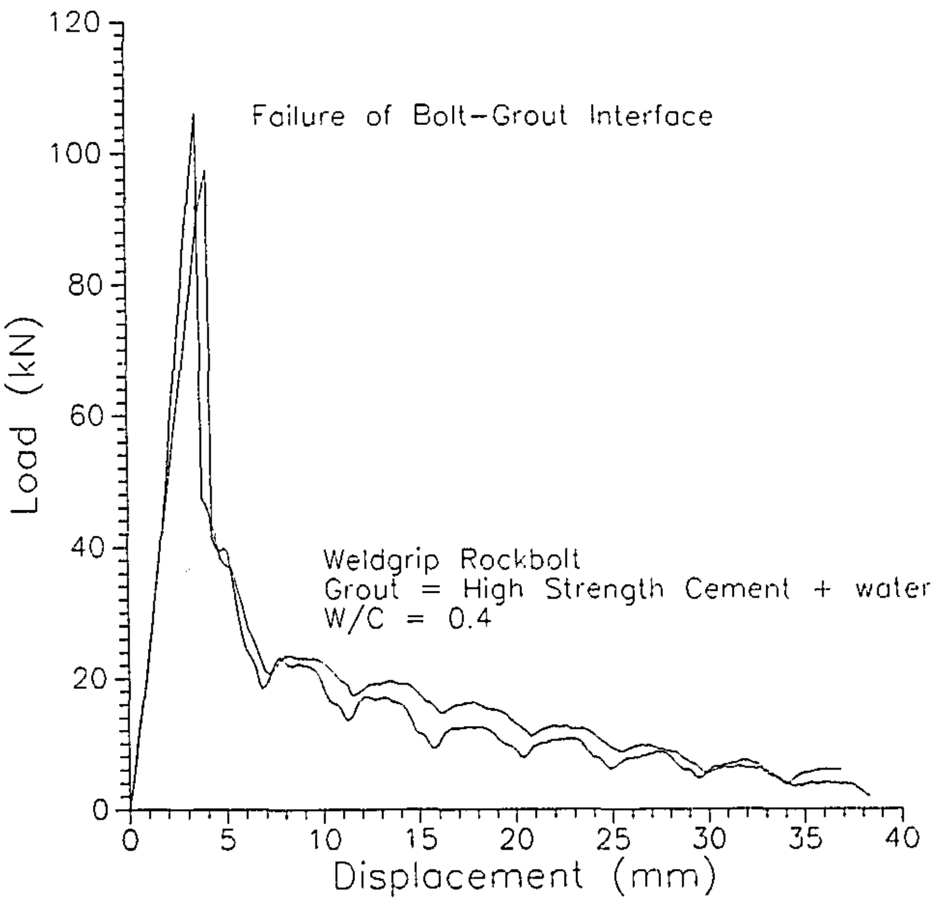


Figure 9.2: Pull out response of Weldgrip bolts grouted with high strength cement grout

9.2.3 Pull Out Tests with Conventional Cement Shotcrete Grout

Shotcrete grout means addition of coarse aggregate in cement grout. The detail about this grout is given in chapter 3. Preparing and mixing of this grout was little more time consuming than conventional grout, however, pouring of shotcrete grout into steel pipe was as simple as its counter part. After 28 days of curing, the samples were pulled out and their response is graphically shown in Figure 9.3. The following are the comments on the experimental results:

1. The ultimate load carrying capacity increases to around 130 kN, which is 43 percent of bolt ultimate breaking load. The behaviour is linear in the elastic zone and has residual load carrying of approximately 30 kN.
2. As usual, threaded portion failed around 90 kN of uniaxial tensile load.
3. The overall load carrying capacity increases by introducing shotcrete grout. The main difference, which can be observed from Figure 9.3, is the residual load carrying response, which did not drop suddenly to 20 kN but first dropped close to 35 kN. It remained static to this load for some times and than dropped to 20 kN and below. This was expected intuitively. As explained earlier, any ordinary means such as increasing size of engraved knobs or adding coarse aggregate in grout, will increase peak load carrying capacity and to some extent post residual load carrying capacity. However, the residual load carrying capacity did no longer stay constantly at the average load carrying capacity i.e. 20 kN. The reason seems to be that coarse aggregate although increases the capacity in the beginning, but when they failed, they completely got crushed and hence, creating a microscopic space (void) between bolt and grout.

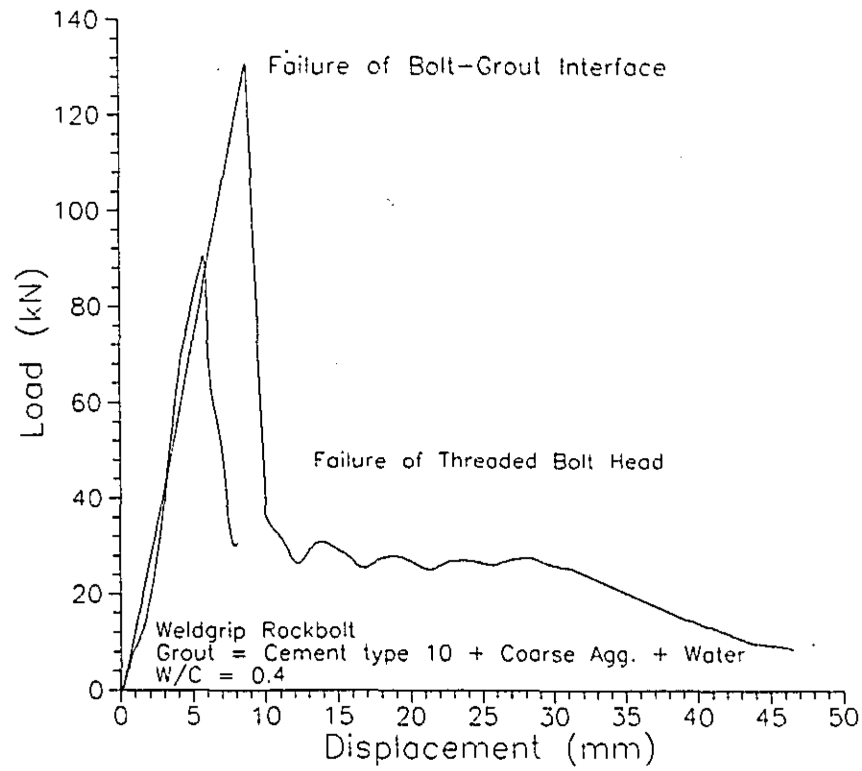


Figure 9.3: Pull out response of Weldgrip bolts embedded with conventional cement shotcrete grout

9.2.4 Pull Out Tests with High Strength Cement Shotcrete Grout

Combination of high strength cement and coarse aggregate resulted into high strength cement shotcrete grout. Weldgrip bolts were grouted and cured for 28 days. These were than pulled out and their pull out behaviour is shown in Figure 9.4. The following are the results and conclusions of the experiment:

1. The peak load carrying capacity is close to 140 kN, which is 47 percent of ultimate tensile load of Weldgrip bolt. The behaviour is linear in elastic region.

2. Post peak residual load carrying capacity is almost identical to the Weldgrip bolts grouted in conventional cement shotcrete grout. In this specific category, load eventually dropped at approximately 5 kN but this drop is more gradual than conventional cement shotcrete grout.

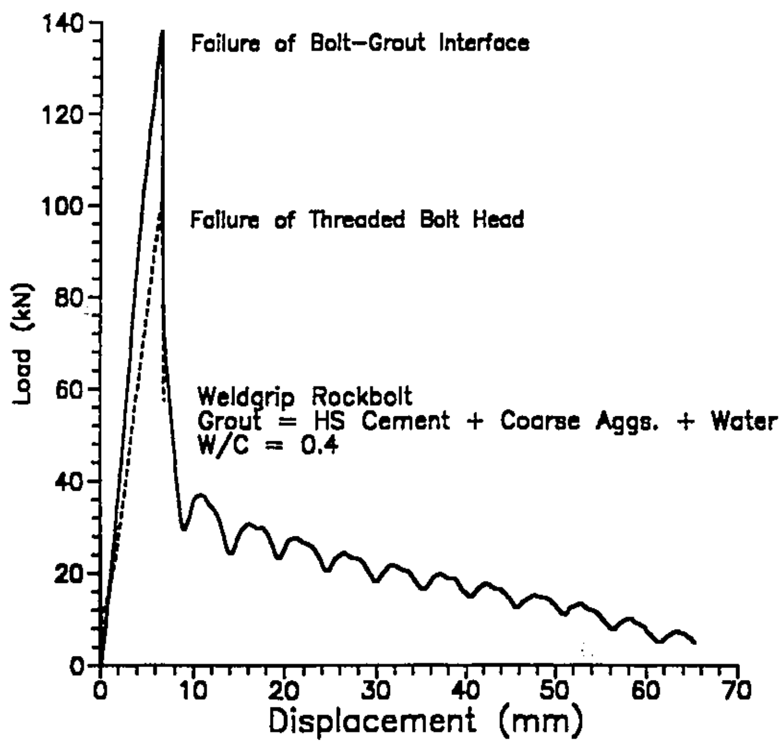


Figure 9.4: Weldgrip bolts grouted in high strength cement shotcrete grout

9.2.5 Comparison of Effect of Grouts on Pull Out Behaviour

A comparison of all those Weldgrip bolts which were failed due to bolt-grout interface debonding, are graphically shown in Figure 9.5. The following are the observation of the overall comparison:

1. The pull out capacity of Weldgrip with conventional and high strength cement is almost equal. This means that the compressive strength of grout does not improve the pull out capacity of Weldgrip bolts. In some of the tests, the threaded bolt head failed, while in other tests, the interface between bolt and grout failed. The bolt itself remained intact and no failure of fibers were observed.
2. The pull out capacity increases with the shotcrete grout. This is observed with both conventional and high strength cement shotcrete grouts. The average pull out capacity with shotcrete grout is close to 130 kN while average capacity of both conventional and high strength cement grouts is 105 kN. Which means that pull out capacity increases with the use of coarse aggregate rather than with the types of cement.
3. It was observed from the experiments that pulled out bolt surfaces were not damaged (scratched, delaminated, brushed) as much as with shotcrete grouts as with the cement grouts.

Table 9.1: Effect of grout types on pull out capacity of Weldgrip bolts

Sample type	$P_{ultimate}$ (kN)	$P_{residual}$ (kN)
Conventional cement	115 ± 5.0	20 ± 3.9
Conventional cement shotcrete	130 ± 3.1	25 ± 4.2
HS cement	106 ± 4.7	5 ± 2.1
HS cement shotcrete	138 ± 4.3	20 ± 5.1

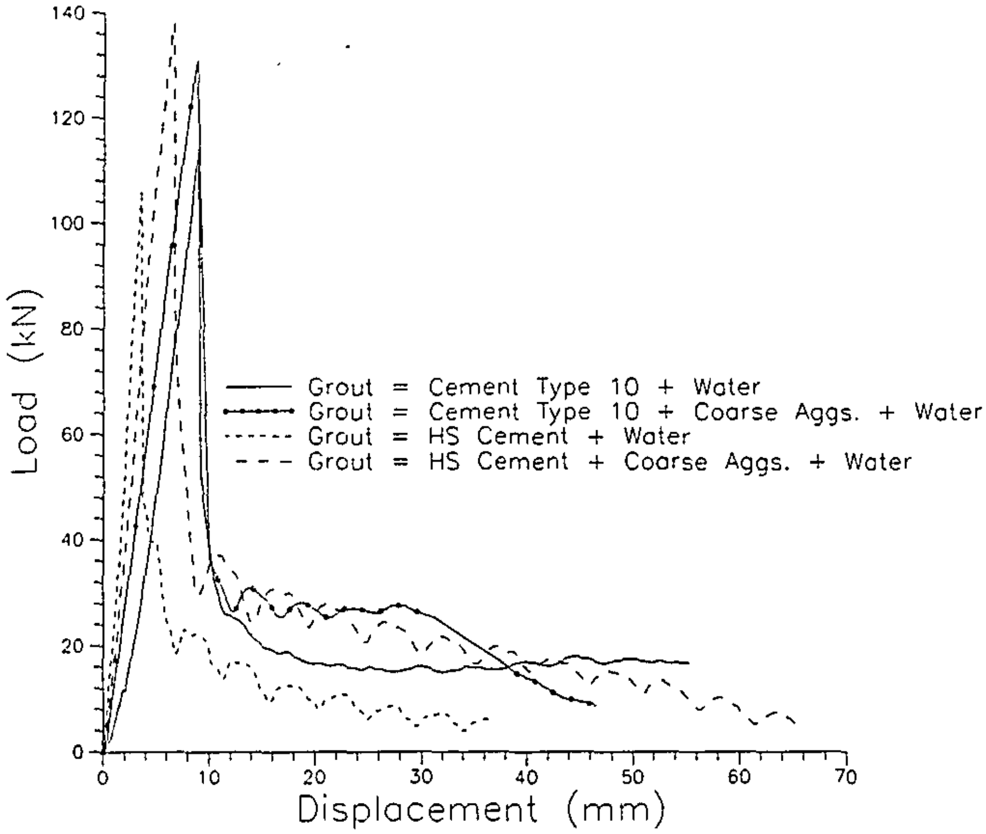


Figure 9.5: Comparison of the effect of different types of cement based grout on the pull out capacity of Weldgrip rockbolts

9.3 Weldgrip Rockbolt Pull Out Tests with Resin Grout

There were primarily two reasons of introducing organic polyester resin as a grout to embed Weldgrip rockbolts. First, to extend the possibilities of applying Weldgrip bolt in salt mines as rockbolts. Cement, because of presence of water in it, can not be used as grout in salt mines. Therefore, viable alternative is resin for fully grouted tendons in salt mines.

Second reason was to find out the critical bond length (maximum grouted length at which bolt break under uniaxial load) of Weldgrip rockbolt. The main advantage of resin grout is that it cures within minutes and can be tested quickly. On the other hand, cement grout takes 7 (at least) to 28 days of curing. It was, however, not found easy to work with the manually mixable resin because of its strong odour.

9.3.1 Short Embedment Length

In the this stage, different embedment lengths, 100 mm (4 inch), 203 mm (8 inch), 305 mm (8 inch), and 406 mm (16 inch) were tested with resin grout. These tests were conducted with the help of hydraulic jack. All the data was collected manually. The results are presented in Figure 9.6. The following are the results and observation of the pull out experiments:

1. All the pull out tests were successful and expected results were obtained. No premature failure due to shrinkage of resin was observed.
2. Test results show that pull out capacity increases linearly with the embedment length.
3. The pull out response of embedment lengths from 100 mm to 200 mm is linear in the elastic region and their slope is more or less identical to pull out test response of conventional bolts. However, in the post-elastic region, their residual load carrying capacity is around 20 kN to 30 kN, which is more than the conventional steel bolts.

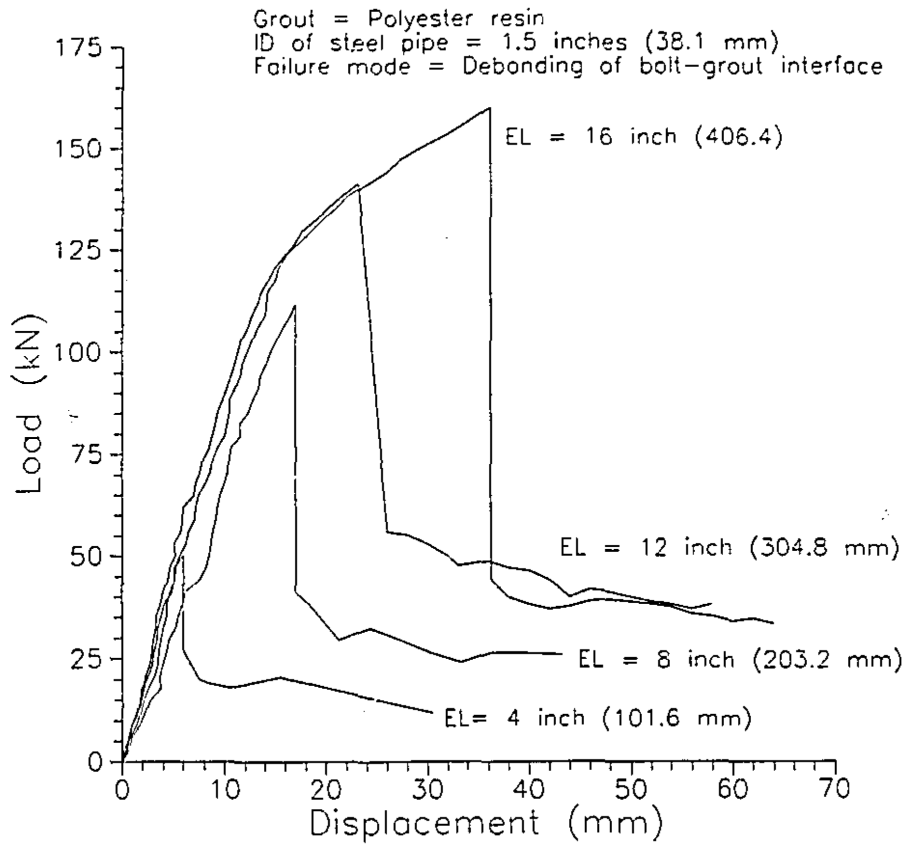


Figure 9.6: Effect of embedment lengths on the pull out capacity of resin grouted Weldgrip bolts

4. As can be seen from Figure 9.7, the response in the elastic region is non-linear for 200 mm plus embedment lengths. The reason is that before pouring the resin in the steel pipe (host medium), tendon was adjusted in the middle of the steel pipe with the help of rubber stopper. This rubber stopper also sealed one end of the steel pipe to prevent the resin from leaking out. It is believed that resin did not cure properly as it does when cured in compressive test sample moulds. Therefore, during the pull out test of 200 mm plus embedment lengths, resin did not behave as brittle as it does under uniaxial compressive tests (see detail in section 3.6.2). Resin, at the loading end,

bulged out and created an unequal load distribution along the embedment length. This phenomenon forced the steel pipe to bend (see Picture 9.1). Hence, a corresponding change in the slope of the pull out test curved was obtained. This behaviour was not expected intuitively.

5. Post elastic residual load carrying capacity of 200 mm plus embedment length samples was found approximately 40 kN.

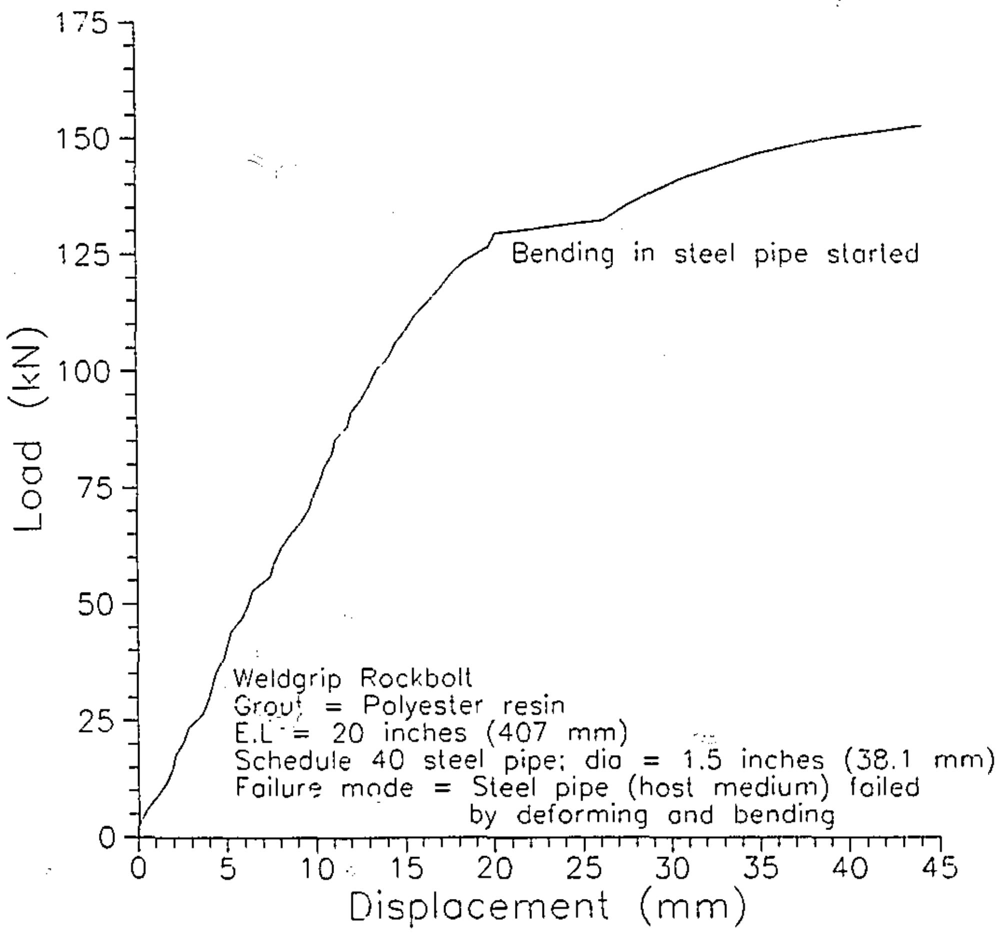
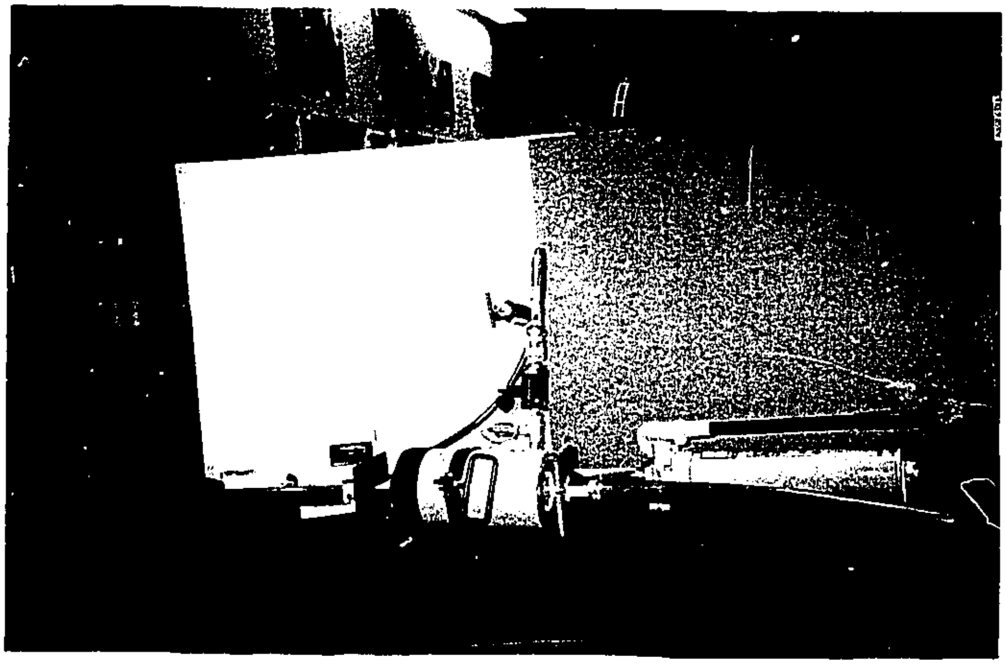


Figure 9.7: Pull out response of Weldgrip rockbolt with an embedment length of 407 mm (20 inch.)



Picture 9.1: Bending of steel pipe during pull out process

9.3.1 Long Embedment Lengths

In this stage, the embedment length or testing length was increased up to 407 (20 inches) and 661.5 mm (27 inches). Whereas the gripping or anchoring length was approximately 1590 mm (65 inch) for both embedment lengths. Schedule 40 steel pipe was used to simulate the host medium. The test results are graphically shown in Figure 9.8 and 9.9. The following are the test results and observations:

1. During the pull out process, it was observed that the steel pipe started to bend from the end close to the applied load. At the same time, resin radially expanded or bulged out at this end. These two factors contributed in the bolt grout interface failure. This is a somewhat premature or incomplete pull out test.

2. The slope in the beginning of the pull out curve is linear. But becomes non-linear with the bending of steel pipe.
3. The obtained ultimate pull out load was approximately 170 kN with both embedment lengths, which was 57 percent of the ultimate breaking load of the bolt.
4. Due to excessive bending of steel pipe, most of the pull out experiment had to stop. While other tests pulled out due to grout-bolt interface debonding. In latter case, an unequal load distribution was seemed to induce along the embedment length of the bolt.

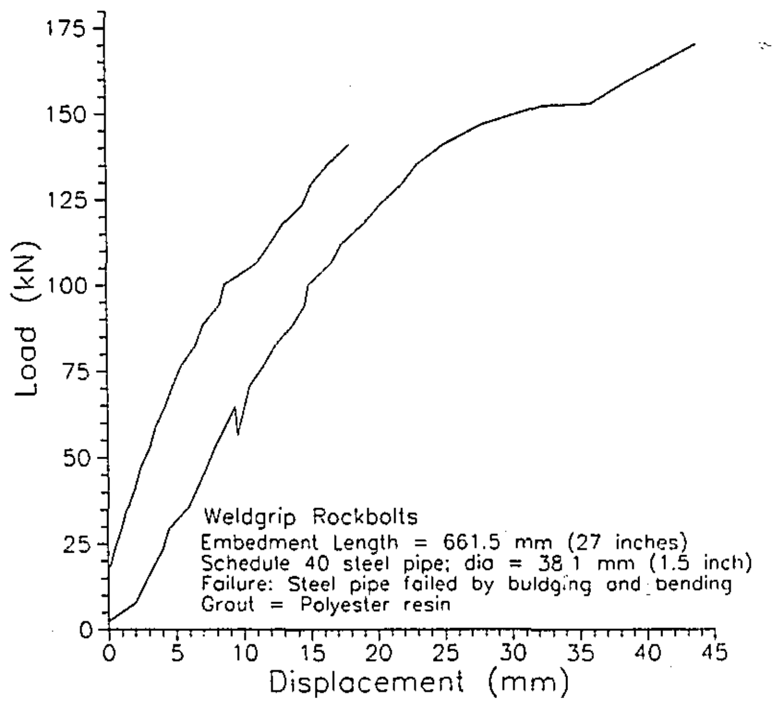


Figure 9.8: Weldgrip rockbolts, grouted in Schedule 40 steel pipe, with an embedment length of 660 mm (27 inch.)

In order to avoid this type of premature failure, schedule 80 steel pipe was introduced to replace Schedule 40 steel pipe. The former is stronger than latter. An embedment length of 661.5 mm (27 inches) was used to grout the bolt and also to compare the effect of stronger host medium. Once again, similar failure for similar reasons were observed, as shown in Figure 9.9. As mentioned before, this behaviour was not observed with the cement grout experiments. This further proved that the bending of the steel pipe was due to inadequate capacity of the resin to sustain the shear load which was developed at the bolt-grout interface. The peak load carrying capacity increased from 170 kN to 190 kN due to stronger host medium (Schedule 80 steel pipe).

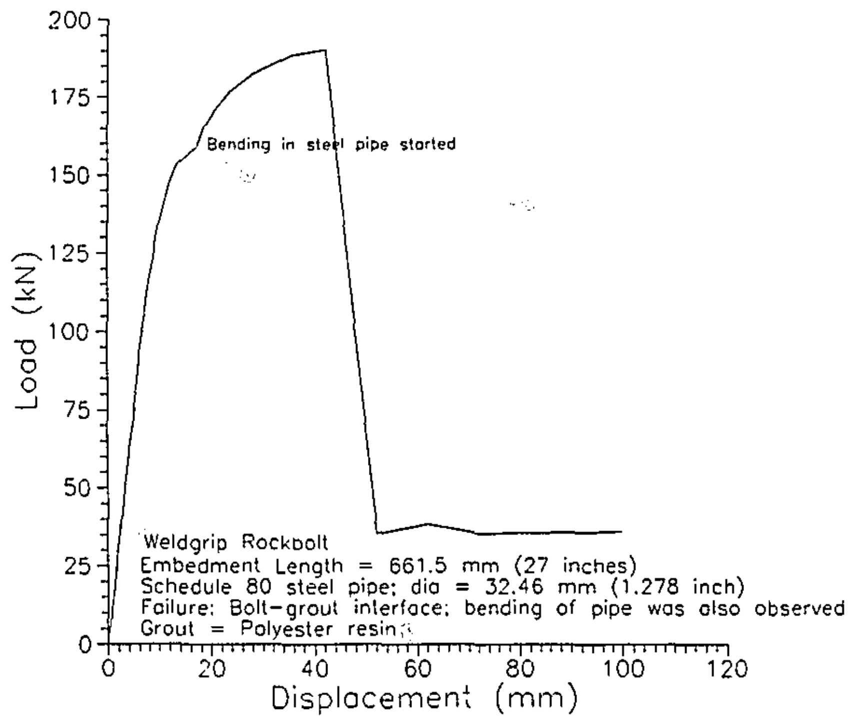


Figure 9.9: Pull out response of Weldgrip rockbolts grouted in Schedule 80 steel pipe with an embedment length of 660 mm (27 inch.)

The effect of host medium pull out response of fully grouted Weldgrip bolts can also be seen from Figure 9.10. The major difference is the slope of the curves. Less bending was observed in case of Schedule 80 steel pipe, therefore it has a less steep slope than

Schedule 40 steel pipe. However, the failure was induced in the former case due to bending of steel pipe, as well.

The slopes of the curves between the load-displacement of the above mentioned pull out experiment of Weldgrip bolts were used to draw the Figure 9.11. The following two linear equations are calculated for two different slopes of the line; see Figure 9.11:

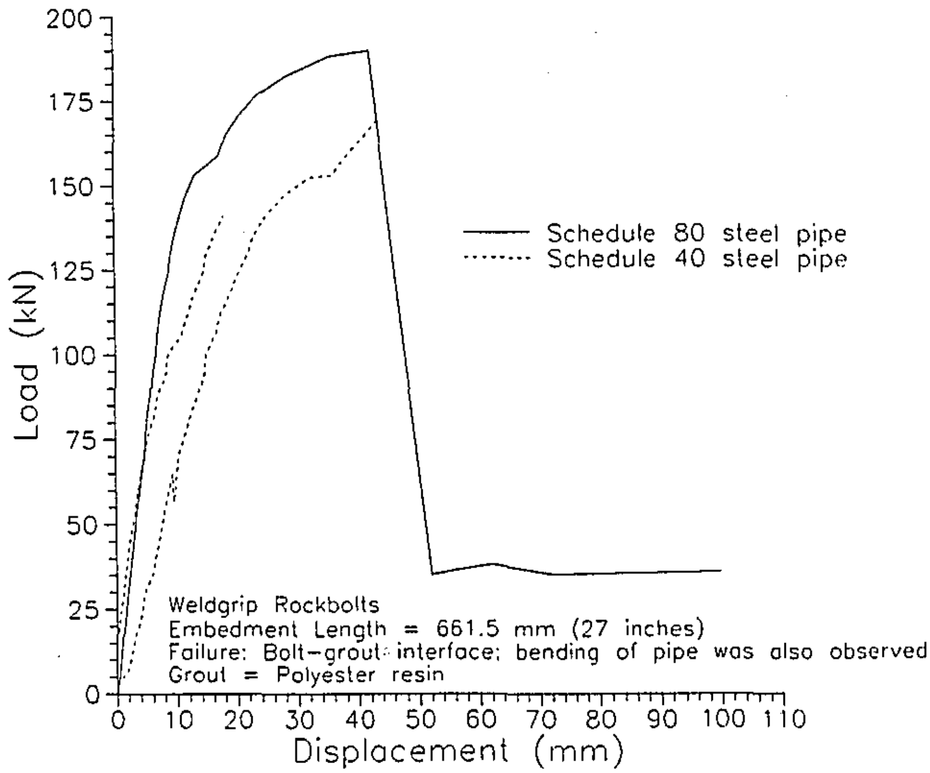


Figure 9.10: Comparison of the effect of host medium (steel pipe) on the pull out behaviour of resin fully grouted Weldgrip rockbolts

For line AB;

$$Pu = 0.5512 \times L_e - 2 \qquad (2)$$

Where 'L_e' is embedment length and it ranges from 50.8 mm to 200 mm, and Pu is the pull out ultimate capacity.

and for line BC

$$Pu = 0.2365 \times L_e + 65.61 \qquad (3)$$

Embedment length 'L_e' ranges from 200 mm to 410 mm.

On the basis of equation 3, the line BC is extended to CD. The ultimate breaking capacity of the Weldgrip bolt is 300 kN, therefore, an embedment length of 1050 mm (40 inches) is required to break the tendon.

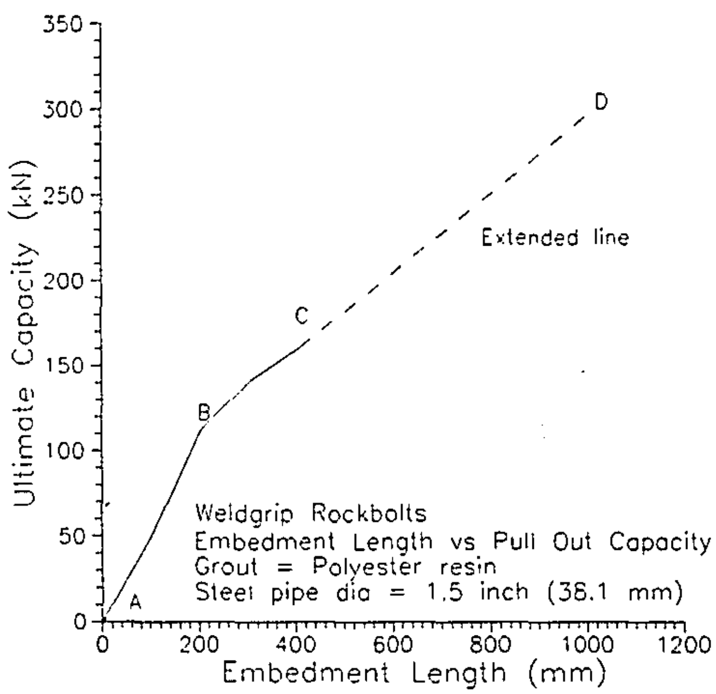


Figure 9.11: Embedded length versus ultimate pull capacity of resin grouted Weldgrip bolts

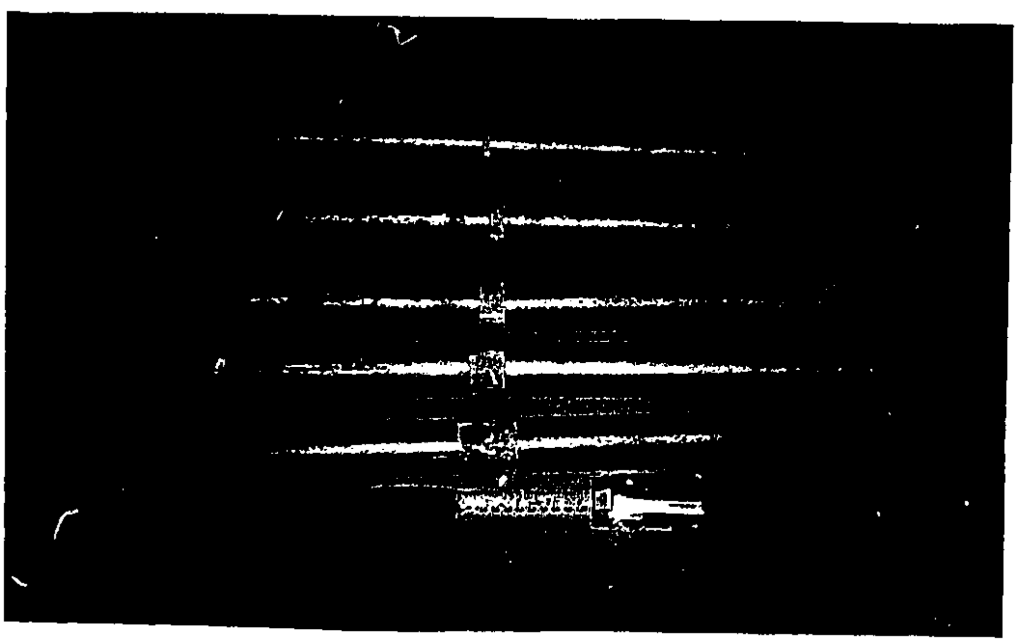
9.4 Shear Test on Resin Grouted Weldgrip Rockbolts

For the shear tests on fully grouted Weldgrip bolts, the following are the parameters considered in the experiments:

a. Stiffness of the Weldgrip bolt. Hollow Weldgrip bolts were used to model less stiff bolts. Solid Weldgrip bolts were also tested to compare their shear resisting performance with hollow bolts.

b. Joint spacing. This was varied from 5 mm to 45 mm (see Picture 9.2). This variable was selected to evaluate the effect of pure shear and the combination of shear and tensile loading.

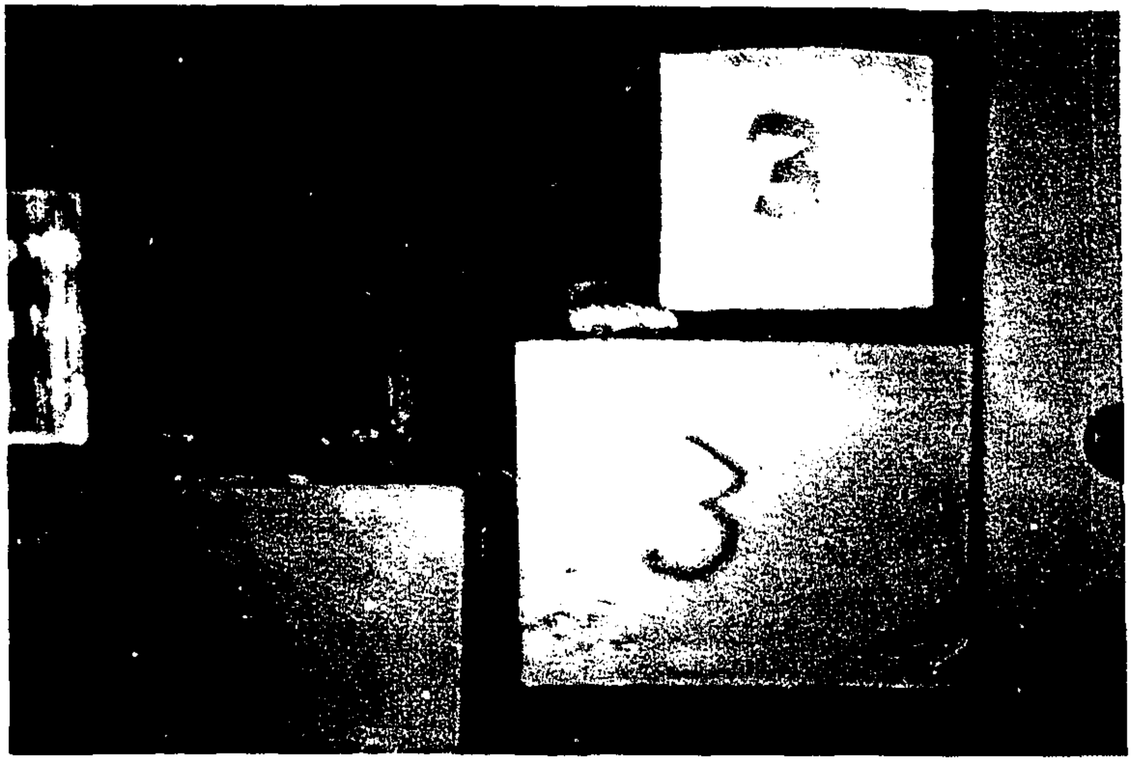
c. Cement grout was also used to see its affect on the shear load and the shear displacement.



Picture 9.2: Various joint spacing

From the Figures 9.12 and 9.13, the following are the results and observations of the shear tests performed on the resin grouted Weldgrip rockbolts:

1. Hollow Weldgrip bolts, due to their low stiffness, resisted more shear load. The tensile mode of failure was more prominent with the increase in joint spacing (see Picture 9.3 for the concept of combination of shear and tensile loading). This is due to the fact that hollow bolts started to bend relatively earlier than solid bolts under combined tensile and shear loading, and thus, they sustained more load.
2. Solid bolts also failed with the combination of shear and tensile loading, however, shear mode of failure initiated with the failure of outer fibers (see Picture 9.4). All solid bolts were partially sheared before bending. This reduced their shear resisting capacity. Therefore, the load resisting capacity of bolts varies close to each other despite the different joint spacing.
3. Some of the solid bolts with a joint spacing of 5 mm sheared at low load, close to 15 kN (see Picture 9.5). These tests were rejected because the absolute shear capacity of bolt itself (without embedding in grout) was obtained around 60 kN.
4. Shear displacement of more than 15 mm was obtained in all the tests. This factor is very useful in designing bolts in a flexible rock environment.
5. The shear load and shear displacement of solid Weldgrip bolt, grouted in cement (see Figure 9.14), was found close to solid bolts embedded in resin grout. The pattern and behaviour of failure were, however, slightly different from its counterpart.
6. These experiments proved that Weldgrip bolts can be used in shear resisting applications in underground mining.



Picture 9.3: Combination of shear and tensile loading

Picture 9.4: Failure initiated from the cutting of outer fibers of the bolt

Picture 9.5: Small joint spacing shear the bolt prematurely

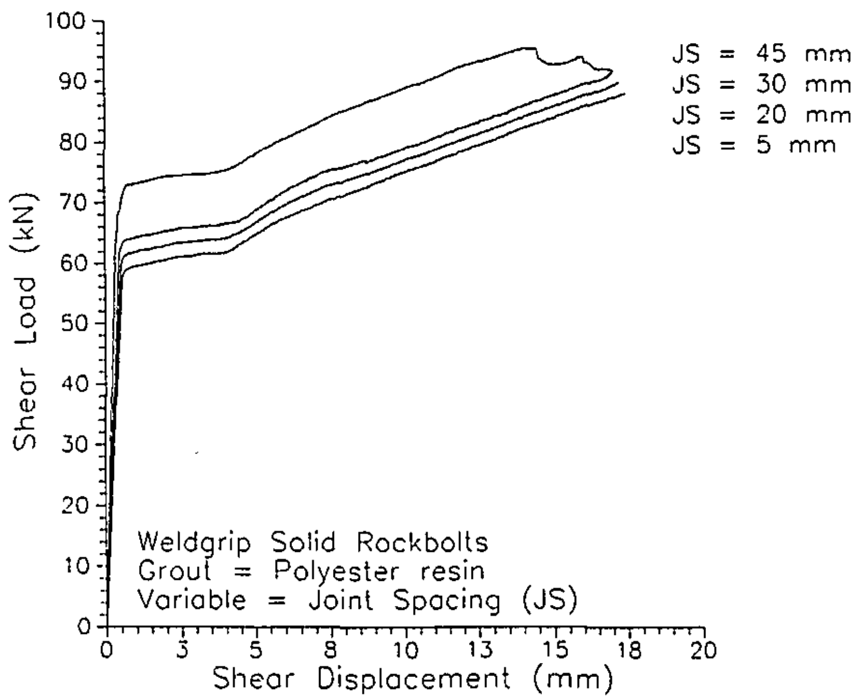


Figure 9.12: Shear test on resin grouted solid Weldgrip bolts

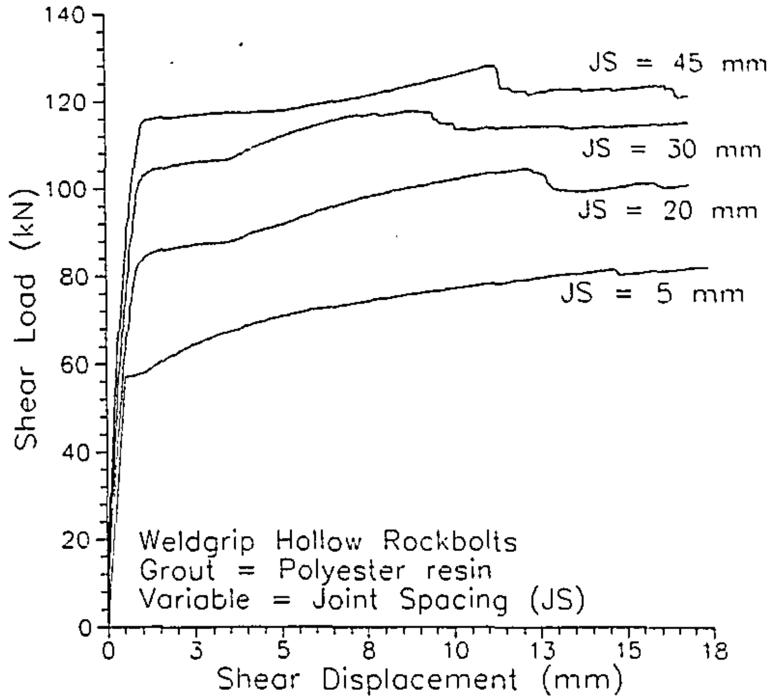


Figure 9.13: Shear test on resin grouted hollow Weldgrip bolts

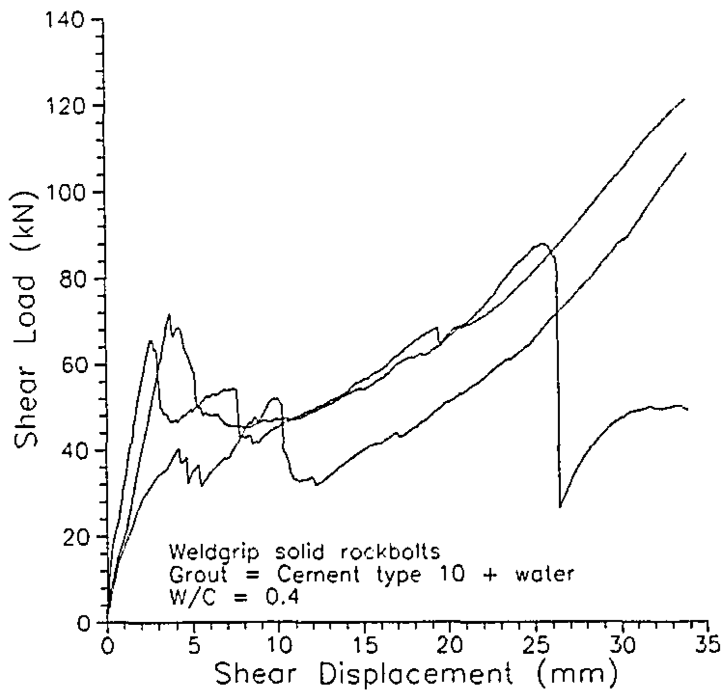


Figure 9.14: Shear test on cement grouted solid Weldgrip bolts

9.5 Discussion

- Weldgrip tendons are rigid bolts that are exclusively developed for underground mining. They have circular cross-section with nominal diameter of 22 mm and a rough surface profile to help improve the bolt-grout interface bonding. A threaded portion is made on one end of the bolt to accommodate nut, face plate and other accessories.
- Laboratory experiments show that Weldgrip tendons have the potential to be used as rockbolt in surface and underground mining. Their pull out behaviour in elastic region is linear and it is almost identical to the pull out behaviour of conventional bolts.
- The ultimate breaking load of Weldgrip bolts is more than 300 kN (breaking strength $\approx$ 790 MPa). A maximum of 200 kN pull out load for an embedment length of 661 mm was obtained in the experiment, which is only 67 percent of the breaking strength.
- Weldgrip rockbolts have more residual load carrying capacity than conventional bolts. This makes them more suitable, since, such a support system will most likely give a warning prior to failure.
- Shear tests on Weldgrip bolts proved that these have potential to be used in shear resisting applications. An average shearing load of 100 kN was obtained in the laboratory experiments.
- Weldgrip rockbolts are cuttable, non-corrosive and non-static in characteristic. So they can be employed in a wide variety of applications in geotechnical engineering.

Chapter 10

Conclusions and Recommendations for Future Research

10.1 Conclusions

The main objective of this research work was to evaluate the mechanical properties of cuttable bolts for underground mining applications. To achieve this objective, research was initiated with a commercial and literature survey of composite cuttable bolts. At the same time, laboratory investigation of different grouting materials compatible with cuttable bolts was also started. Conventional fully grouted flexible steel cable was also included in the research program, because its behaviour could be generalized for flexible composite tendons such as Arapree and Polystal.

Grouting Materials

- High strength cement grout (HSCG) was found to be superior in performance over conventional cement grout (CCG). The uniaxial compressive strength (UCS) was found to be around 90 MPa for the former and 50 MPa for the latter. Approximately 50 MPa of UCS was obtained after two days of curing for HSCG, whereas, the same value for CCG was resulted after 28 days of curing. Figure 10.1 shows the gain in strength by HSCG and CCG against time (days). The modulus of elasticity of HSCG, however, did not increase linearly with UCS and ranged between 40 - 50 GPa. While in CCG, this value ranged between 25 - 35 GPa. These results ensured the ductile behaviour of HSCG.

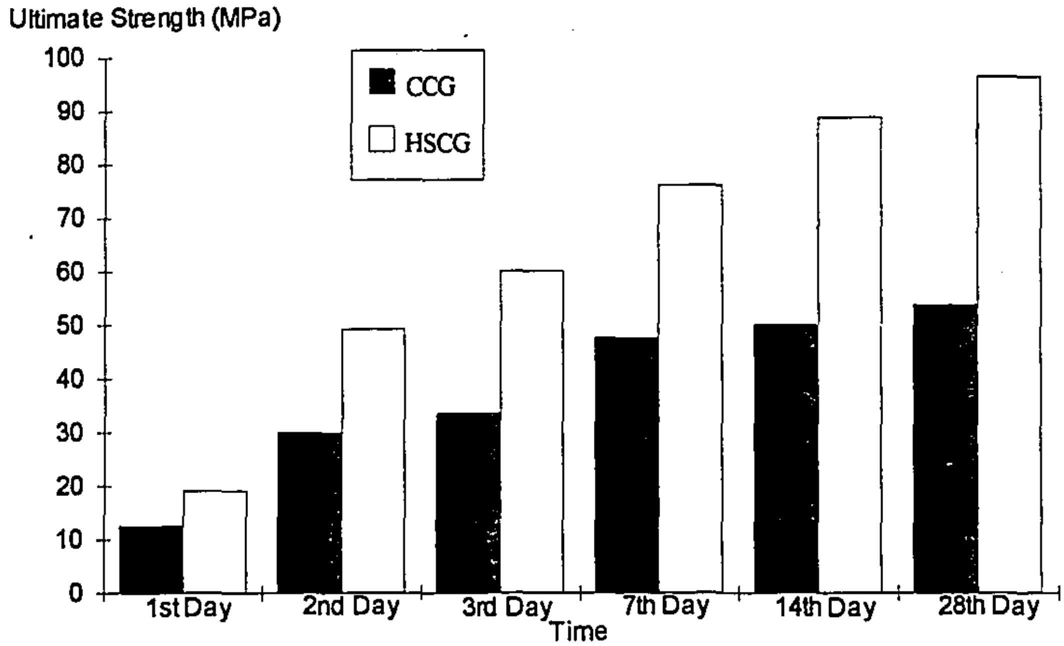


Figure 10.1: Comparison of gain of ultimate strength by High Strength Cement Grout (HSCG) and Conventional Cement Grout (CCG) against time (days)

- The addition of coarse to fine aggregates in cement (shotcrete) grout did not seem to improve its mechanical properties. Experiments showed that fine aggregates do not need to be added to a shotcrete mix. It made mixing tedious and reduced the workability. The presence of coarse aggregates in cement grout is concluded to increase the frictional resistance between cable and grout during the pull out process.
- A manually mixable resin was introduced and tested, for the first time, to obtain its mechanical properties. The advantage of this system over conventional sausage (cartridge) grout is that it does not require the spinning of bolt. Therefore, this grout can be used with good quality control for small embedment length. The uniaxial compressive strength and modulus of elasticity were found, respectively, to be 76 MPa and 11 GPa.

Conventional Steel Cable

The laboratory investigations on fully grouted seven wire conventional steel cable considered a number of parameters relating to its performance not previously considered. The main purpose was to provide insight into the interaction between cable and grout.

- The effect of the hole diameter of the loading plate was evaluated, although this test was limited to the laboratory scale only. It has been shown that the significance of the strength and stiffness of both grout and host medium is considerable. The radial confinement exerted by the host rock mass was also modelled in the laboratory. The interaction between the mechanical properties of the grout and the host medium, together with the confining pressure, appeared to affect the performance of the fully grouted cable significantly.
- The effect of the rate of displacement on fully grouted cables showed that, within the experimental range of loading rates, the performance of cable is independent of such rates. This indicates that cable support systems should perform effectively under dynamic loading, such as from blasting and rockbursting.
- A linear relation was observed between cable embedment length and pull out capacity. The critical length of cable bolt was found to be approximately 80 cm. A comparison of high strength cement grout and conventional grout was also made, which showed that the former has 45 % more bonding strength than the latter. The ductility of the system, however, remained unaffected. Different mixes of shotcrete were also tested. A mix containing cement and coarse aggregate was found to be superior than conventional grout.
- The shear resisting capacity of fully grouted cables was found to reduce with increase in joint spacing. Cable, being flexible, was not found to be effective in preventing shear

displacement. Cables started to bend after a shear displacement of a few millimeters, although they effectively sustained the shear load.

Composite Tendons

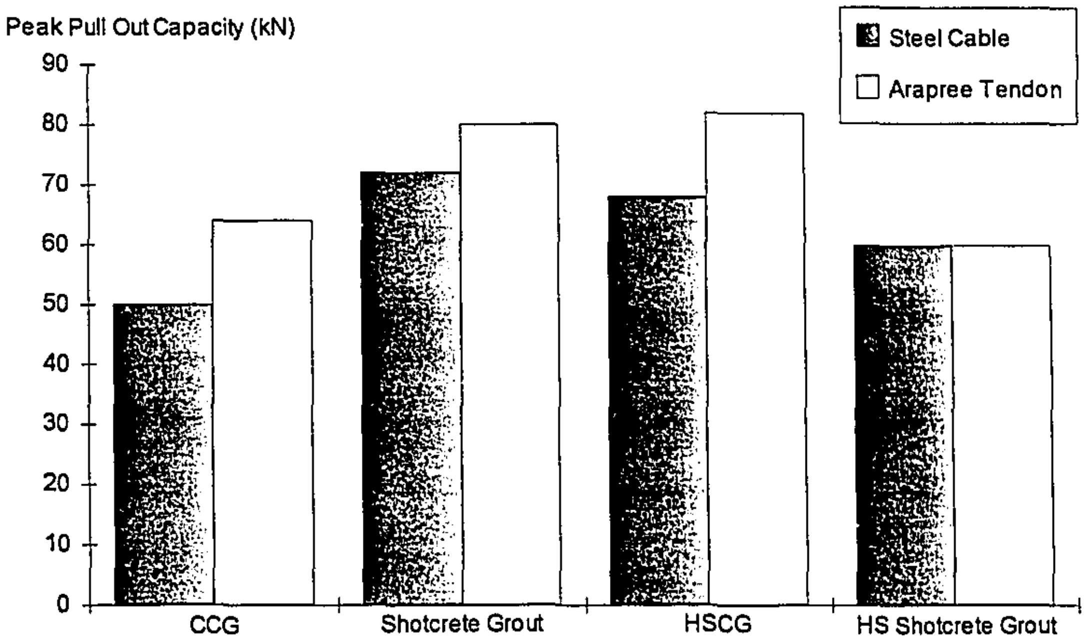
In order to identify cuttable support systems in underground and surface mining, a comprehensive commercial and literature survey was undertaken. From various cuttable support systems, such as chemical grouts, shotcrete, etc., uniaxial tensile load bearing composite tendons were found to be the most promising for interior rock support. Sixteen different types of composite tendons were selected from a survey of approximately one hundred composite manufacturing companies. A preliminary laboratory investigation was conducted to identify composite tendons which could have compatibility with mining requirements in considerations such as safety and economy. Two composite tendons, Arapree and Weldgrip were selected for detail testing.

Arapree Tendon

The conclusions regarding fully cement and resin grouted Arapree tendons are given below. Emphasis is given to the behaviour of Arapree within the elastic limit, since this is most often used in support systems design.

- The laboratory investigation proved that Arapree tendons with cement grout have the potential to be employed in mining applications and that such composite materials are expected to be the ground support of the future.
- In mining applications, cable bolts and rockbolts are designed considering their elastic limit under tensile loading. Arapree as a cable bolt and/or rockbolt behaves like steel

cable bolt in the elastic limit (see Figure 10.2). Therefore, Arapree tendon systems should be appropriate to support the rock mass in place of steel cables.



(CCG = conventional cement grout; HSCG = high strength cement grout; HS = high strength)

Figure 10.2: Performance of Arapree and steel cable in different types of grouts within the elastic limit

- The post-elastic limit of Arapree tendons is less than steel cable but is more than that of steel rockbolts. The residual capacity of fully cement grouted Arapree tendons is around 50 kN. This means that the rock mass supported by Arapree tendons should give warning prior to failure.
- The capacity of fully grouted Arapree tendons can be improved by coupling two Arapree tendons; the twin parallel or twin zig-zag configurations. Their performance within the elastic limit is shown in Figure 10.3.

- Single and twin parallel Arapree tèndons embedded in resin grout fail prematurely with a very low peak load carrying capacity. Figure 10.4 shows their performance in the elastic zone, with the pull out characteristics of Arapree grouted in cement for the purpose of comparison. Therefore, Arapree is not recommended to be used alone as a reinforcing element with resin grout.
- The twin zig-zag configuration of Arapree tendon sustained the expected load. This was due to the improved interlocking between tendon and grout. Failure was initiated due to the failure of the manually manufactured knots.
- The shear load carrying capacity of Arapree tendon was found to be relatively low. This is because of the flexibility of tendon that led to the gradual shearing of fibers rather then distributing the shearing load on the entire cross-sectional area of the tendon.

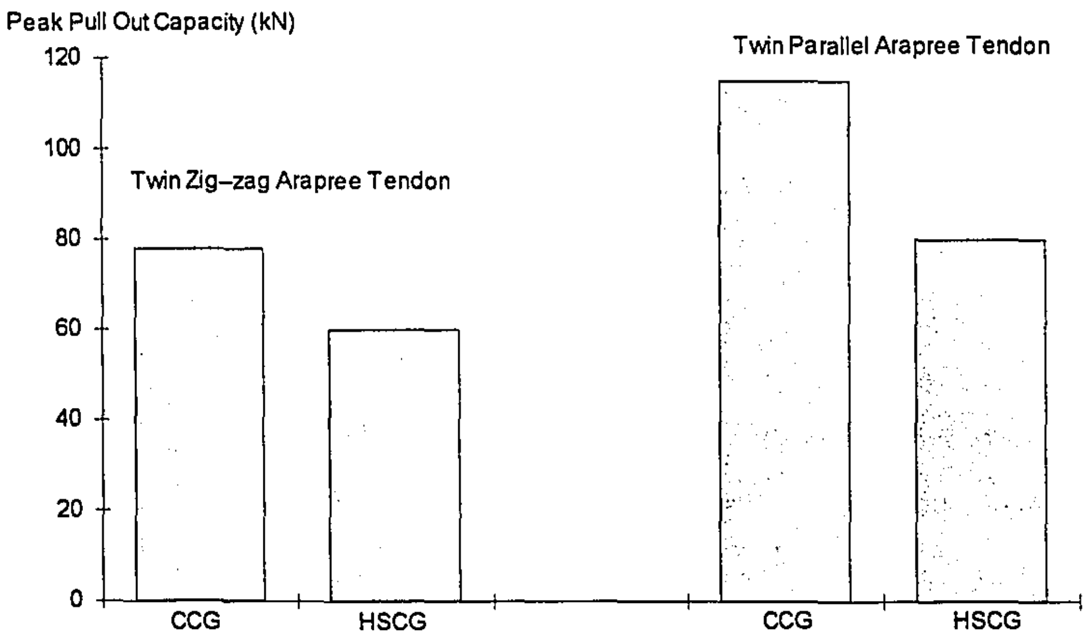


Figure 10.3: Performance of different configuration of Arapree tendons in different grouts within elastic limit

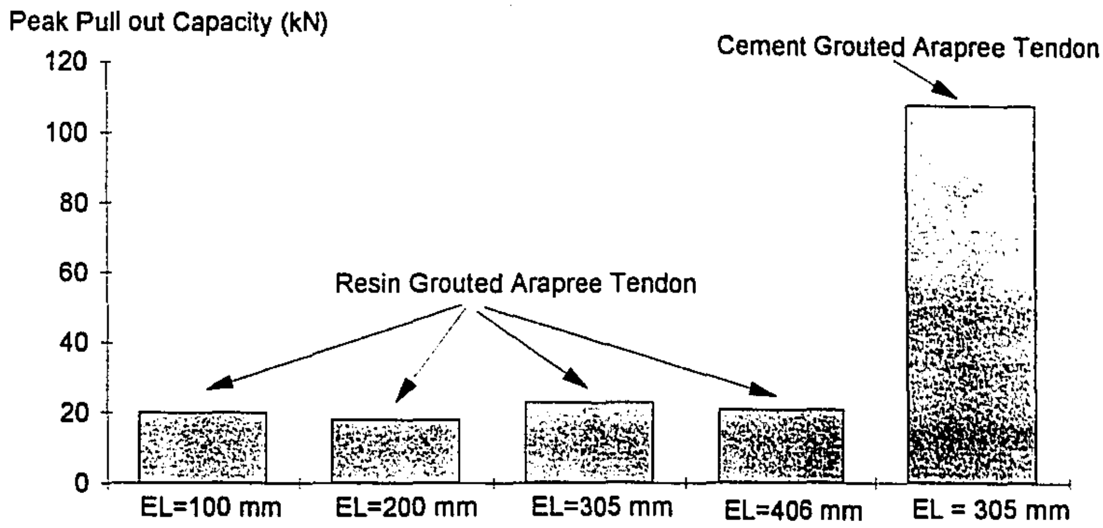


Figure 10.4: Performance within the elastic limit of different Embedment Lengths (EL) of Arapree tendons embedded in resin and cement grout

Weldgrip Rockbolt

A laboratory experimental evaluation was made of the Weldgrip rockbolt. Once again, emphasis was given to the performance of these bolts in the elastic region.

- Laboratory experiments clearly indicated that Weldgrip rockbolts have the potential to be used as rockbolts in surface and underground mining. Their pull out behaviour in the elastic region is linear and it is almost identical to the pull out behaviour of conventional bolts; see Figure 10.5.
- Weldgrip rockbolts have more residual load carrying capacity than conventional bolts. This makes them more suitable to sustain brittle rock failures, since, such a support system will most likely give a warning prior to failure.

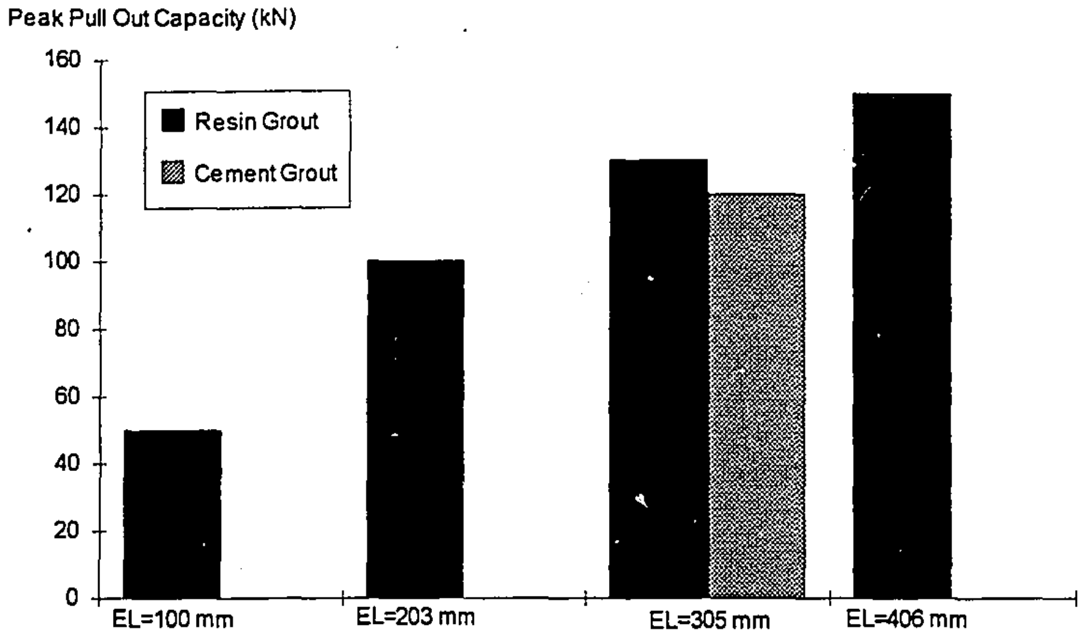


Figure 10.5: Pull out capacity of resin and cement grouted Weldgrip rockbolts within elastic limit

- The ultimate breaking load of Weldgrip bolts is more than 300 kN (breaking strength ≈ 790 MPa). A maximum of 200 kN pull out load with an embedment length of 661 mm was obtained in the experiment, which is only 67 percent of the bolt breaking load.
- Shear tests on Weldgrip bolts showed that they have the potential to be used in shear resisting applications. An average shearing load of 100 kN was obtained in laboratory experiments. A relationship between discontinuity spacing and effectiveness of solid and hollow Weldgrip bolts were evaluated and results are shown in Figure 10.6.

In summery, tendons from composite materials displayed superior performance than existing steel counterparts. Composite materials are at present expensive. It is foreseeable, however, that as production volume and competition increases, then costs will eventually drop to acceptable levels.

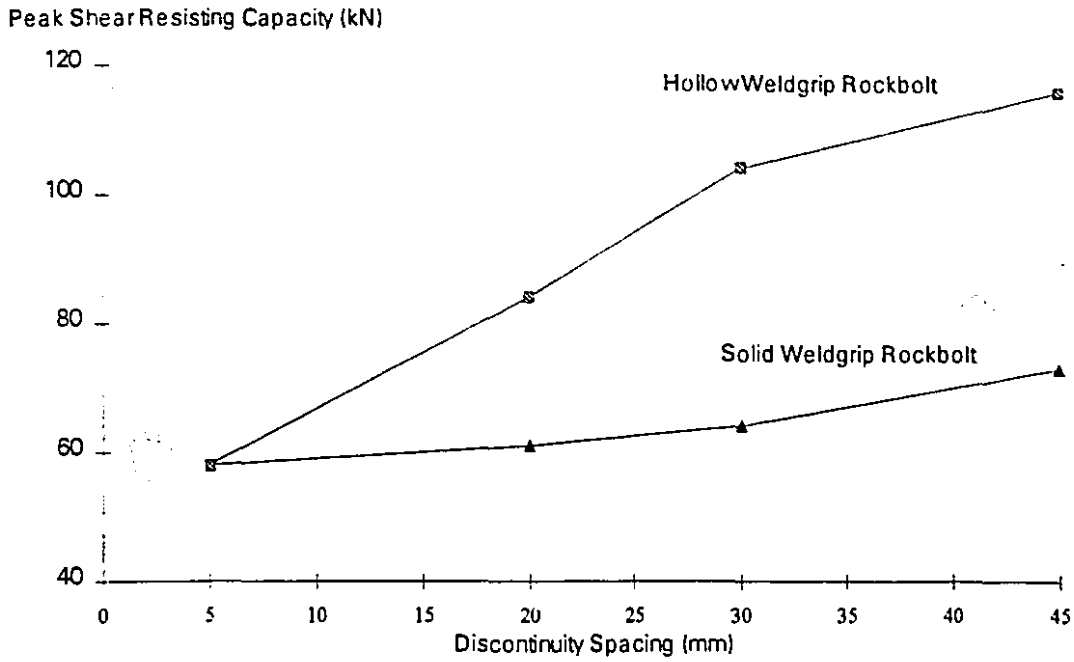


Figure 10.6: Effect of discontinuity spacing on the shear resisting capacity of solid and hollow Weldgrip rockbolts

10.2 Recommendations for Future Research

- An important concern in every field where composite products are gaining acceptance is the lack of available design criteria and, even more importantly, of standard test methods and equipment. Mining R & D should also focus on initiatives to introduce, develop and produce design guidelines for composites. Such initiatives should move the composite manufacturers towards mining application needs.
- An embedded light wave transmitter fiber optic in a composite bolt could continuously monitor its performance. This technology is simple to introduce in composite tendons at the time of their manufacturing, therefore, it is recommended that this technique should to be investigated.
- Field trials of composite tendons are required to confirm some of the laboratory results reviewed in this study.

- The flexibility of composite tendons can be increased or decreased at the time of manufacturing. This work suggests that a "yieldable" bolt could be achieved and this bolt which would meet mining requirements could be achieved by making use of this technological feature.
- Performance of composite bolts in the presence of ground water should be the subject of future research.

Statement of Originality

Extensive experimental studies have been conducted to investigate the mechanical behaviour of grouting materials, fully grouted steel cables and different types of composite material tendons. The grouting materials used in the specimens included conventional cement, high strength cement, shotcrete grout and organic resin. The main variables such as hole diameter of loading plate, strength and stiffness of both grout and host medium, radial confinement pressure on cable-grout interface, rate of loading, etc., were investigated to study the bond strength of fully grouted steel cables. The pull out behaviour of sixteen different types of composite material tendons have been investigated to study their compatibility with mining requirements.

New methods of testing were developed to study the pull out behaviour and shear resisting capacity of steel and composite material tendons. The pull out testing technique enabled experimentation on different types of composite material tendons. The experimental setup for shear testing enabled the investigation of the influence of discontinuity spacing on shear resisting capacity of fully grouted steel and composite material tendons.

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